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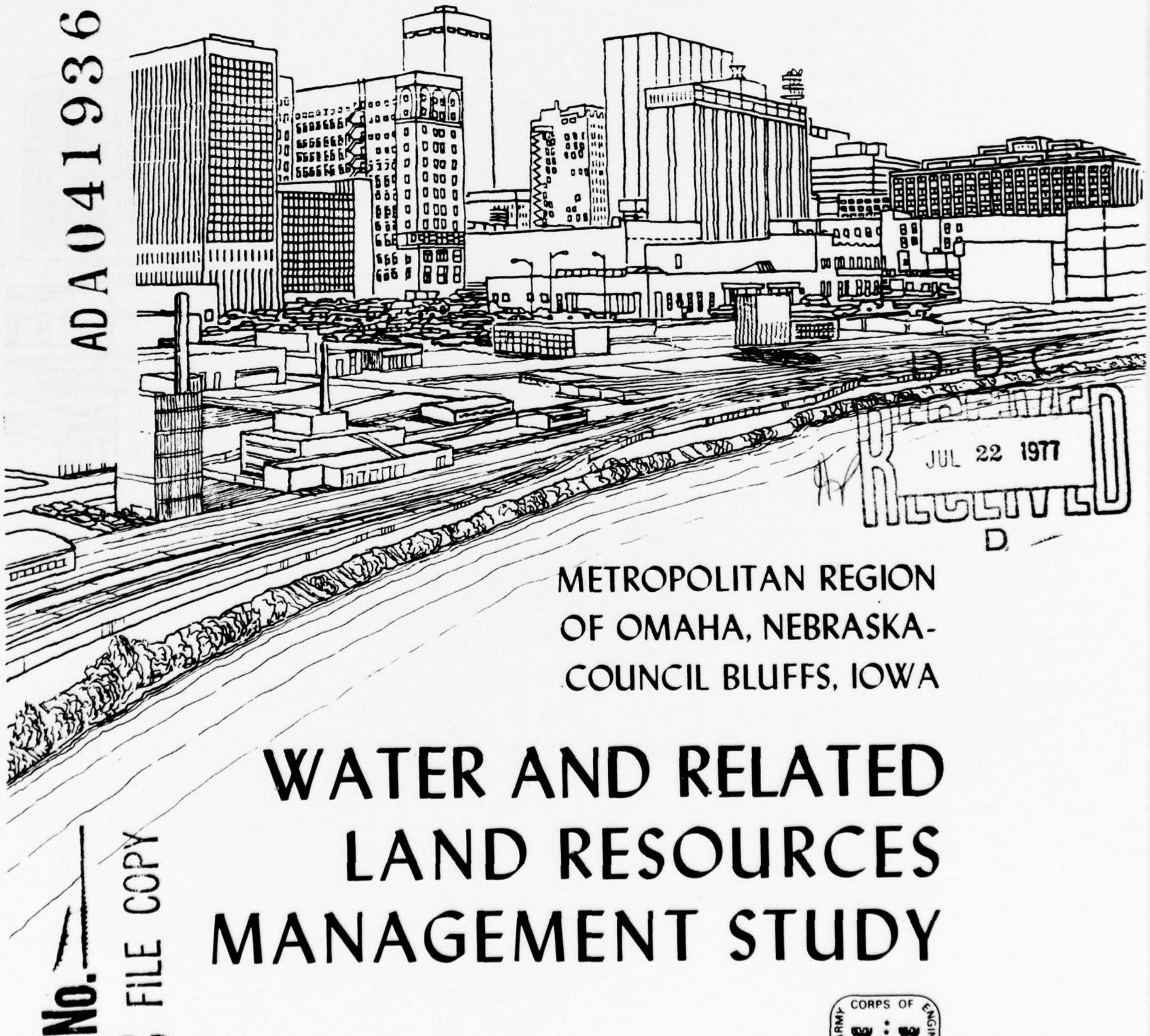
REVIEW REPORT ON THE MISSOURI RIVER AND TRIBUTARIES



METROPOLITAN REGION
OF OMAHA, NEBRASKA-
COUNCIL BLUFFS, IOWA

WATER AND RELATED LAND RESOURCES MANAGEMENT STUDY

JUNE 1975

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Annex J. Abatement of Pollution from Combined Sewer Overflows.

ALTERNATIVE PLANS FOR ABATEMENT OF POLLUTION FROM COMBINED SEWER OVERFLOWS - OMAHA, NEBRASKA

HARZA ENGINEERING COMPANY

October 1974

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SUMMARY LETTER

HARZA ENGINEERING COMPANY CONSULTING ENGINEERS

October 1, 1974

Omaha District
Corps of Engineers
6014 U.S. Post Office
and Court House
Omaha, Nebraska 68102

Subject: Alternative Plans for Abatement of Pollution from Combined Sewer Overflows - Omaha, Nebraska

Gentlemen:

We are pleased to submit our final report on the formulation and evaluation of alternative plans for abatement of pollution of the Missouri River by combined sewer overflows in Omaha, Nebraska. As part of this assignment, we have also assessed septic tank and high water table problems in the Carter Lake area and the impact of the Riverfront Development Program on the existing sewer system. One of the alternatives presented herein is to become a part of the regional water quality management plan currently in preparation.

The planning program was carried out in two phases. Phase I consisted of the formulation and preliminary evaluation of a number of alternative concepts for solving the combined sewer overflow problem. Twelve alternative concepts were selected as the most feasible from 30 possibliities considered. From these, the City of Omaha and other concerned agencies selected five alternative plans for detailed study in Phase II.

The Combined Sewer Pollution Problem

The Omaha-Missouri River Sewerage System serves the 22,000-acre portion of Omaha that drains to the Missouri River. Dry-weather sanitary sewage from the ten major sewer service areas is discharged to an interceptor system leading to the Missouri River Treatment Plant on the south side of Omaha. The plant is designed for primary treatment (solids removal) with a capacity of 72 mgd. Planned expansion will include secondary (biological) treatment.

During periods of rainfall or snowmelt, flow in the combined sewers often exceeds the capacity of the interceptor and overflows occur at 22 points along the riverfront. Overflows occur about 50 times a year resulting in an annual discharge to the river of about five billion gallons of mixed sanitary sewage and storm water, eight million pounds of suspended solids, and four million pounds of BOD. Additionally, frequent overflows of sanitary sewage are caused by mechanical breakdowns in the interceptor system.

Reduction or elimination of this pollution source is required under State and Federal law.

Prior to formulation of alternative plans for resolution of the combined sewer overflow problem, it was necessary to establish the approach for dealing with wastewater and storm runoff from unsewered portions of the Carter Lake area and to assess the possible impacts of the Riverfront Development Program on the existing sewerage system and on alternative plans.

Septic tanks, used for residential wastewater disposal in most of the Carter Lake area, function poorly, supposedly because of a high water table. We conclude that the soils in the area are unsuitable for septic tanks, irrespective of ground-water levels and that the area should be served by separate sanitary sewers. This area then will have no effect on alternative plans for combined sewer pollution abatement.

We conclude that Riverfront Development Program projects in combined sewered areas generally should be served by combined sewers. These projects will cause little change in storm runoff so will not affect alternative plans for combined sewer pollution abatement. However, some RDP projects may preempt land areas designated for structural facilities in some alternative plans.

Alternatives for Combined Sewer Pollution Abatement

All combined sewer pollution abatement concepts considered, with the exception of sewer separation, include storage of the mixed storm runoff and sanitary sewage to allow treatment at a rate of flow considerably less than the peak flow rate in the sewers. In most cases, the combined sewage would be treated at the existing Missouri River Sewage Treatment Plant. When secondary treatment is added to the plant, its dry-weather treatment capacity can be supplemented with special wet-weather facilities to accommodate the additional load.

Since the specific effluent quality requirements for treated combined sewage have not as yet been established, we have considered in our studies three levels of treatment, from secondary to background river water quality. Similarly, a range of design recurrence intervals of from one to ten years was considered. A 5-year design recurrence interval together with secondary-level treatment was used for comparing alternative plans. If the design criteria for combined sewers, when they are defined by the EPA, should fall outside these constraints, alternatives that were rejected because they could not meet the minimum treatment and performance criteria set for this study should be reconsidered.

Five of the alternative concepts were selected by the City of Omaha and other concerned agencies for further detailed study. Summary cost estimates for the five selected alternative plans, for 5-year design recurrence interval and secondary treatment, are shown in Table A. Performance and costs of these alternative plans for 1, 2, 5, and 10-year design recurrence intervals and for higher levels of treatment were also evaluated and are given in the body of the report. The relative merits of the alternatives remain essentially the same irrespective of the given recurrence interval and level of treatment.

The five alternatives that were studied in detail are discussed below:

Diked Storage Along Levee. Combined sewer overflows at the existing outfalls would be stored in open reservoirs constructed by diking off elongated areas on the river side of the existing interceptor sewer system for treatment at a wet-weather biological treatment facility at the Missouri River Plant. Overflows would be retained in storage for a maximum of 20 days and would be mechanically aerated while in storage to minimize the possibility of odors.

Five reservoirs occupy approximately 300 acres along the riverfront. The reservoirs would be located near Minne Lusa, Grace St., Leavenworth, Missouri Avenue, and Monroe Street. The reservoirs would be about 10 to 15 ft. deep and would be completely emptied and cleaned during the winter months. The added encroachment in the river floodway would not significantly raise flood water levels.

This is the least costly alternative. It has the advantage of using the existing flood control levees for one wall of the reservoirs and is amenable to staged construction. Being on the river side of the levees, visual impact of the reservoirs would be low. Nevertheless, this alternative is likely to have the poorest public acceptance.

Deep Tunnel to Ground Level Storage. In this plan, the elevation of much of the combined sewer service area would be used to provide energy for gravity conveyance of combined sewer overflow through a tunnel to a low-cost diked reservoir across the Missouri River north of Council Bluffs. The overflows would be diverted to drop shafts and to a tunnel beneath the existing interceptor. The tunnel would be constructed in sound rock formations several hundred feet below the surface. As in the previous alternative, stored combined sewage would be pumped to the existing interceptor system for treatment at a wet-weather facility at the Missouri River plant.

Since the only visible facility would be located in a currently agricultural area and would have a low profile, public acceptance should be good and environmental impact minimal.

Excavated Storage - Deep Tunnel to Ground Level Storage. This alternative is a modification of the preceding alternative in which the northern and southern zones of the combined sewer area would be dealt with individually. Low cost, shallow excavated reservoirs in the Carter Lake area would be used for storage of overflows from the northern zone. A tunnel to ground level storage located across the River, south of Council Bluffs would serve the southern zone.

The dual concept alternative is somewhat less costly than the single concept tunnel alternative, but it would be more disruptive. Public acceptability for this alternative would be reduced because of the open reservoirs in proximity to the urban area. The reservoir sites also may be in conflict with proposed Riverfront Development project.

Deep Tunnel to Mined Storage. This alternative utilizes a smaller diameter tunnel than the preceding deep tunnels and flows from North to South, discharging by gravity to a mined storage reservoir located 500 to 600 feet below ground. Diversion structures and drop shafts at the outfalls would allow diversion of the overflows to the tunnel and the storage chambers. The initial storage chamber would be a settling chamber for removal of settleable solids. Overflows from this chamber would be aerated in the main chambers. The stored overflows would be lifted to the Missouri River Plant for treatment. Solids and grit would be pumped separately to the surface for disposal.

Although the cost for this alternative is relatively high, the storage facilities are located underground and not visible to the general public, making this plan the most environmentally acceptable alternative.

Deep Tunnel to Papillion Creek. A modification of the previous alternative would utilize a deep tunnel extending from the mined storage facilities to the Papillion Creek area. The objective would be to convey the overflows to a central location for potential disposal and final treatment as required. This plan could also eliminate the proposed secondary expansion of the Missouri River treatment plant by conveying the primary effluent with the deep tunnel to the Papillion Creek area.

This plan will require further evaluation as part of regional wastewater management plans that are being considered for abatement of pollution.

Conclusion

Of the five alternatives, diked storage along the levee would have the lowest capital and operation and maintenance costs. Also, it could be implemented sooner, would require less construction time and is more amenable to staged construction than the alternatives. However, this alternative would have the poorest social and environmental acceptability, while the deep tunnel

with underground storage, the highest cost alternative would have minimal adverse impact. Deep tunnel to ground level storage, excavated or not, appears to be the best compromise of cost and social and environmental acceptability.

We trust that the material in this report will provide an adequate basis for selection of an alternative plan and for further studies. We look forward to continuing our participation in this important and challenging planning program.

Very truly yours,

J. C. Ringenoldus
Project Director

Table A

SUMMARY OF COSTS OF FIVE

ALTERNATIVE PLANS STUDIED IN DETAIL

Alternative Plan	Construction Cost	Annual Operation and Maintenance	Total[1/] Present Worth
	Millions of Dollars		
Diked Storage Along Levee	$ 71	$4.7	$144
Deep Tunnel to Ground Level Storage	145	5.1	222
Excavated Storage-Deep Tunnel to Ground Level Storage	111	5.1	187
Deep Tunnel to Mined Storage	179	6.1	270
Deep Tunnel to Papillion Creek	192[2/]	5.6[2/]	276

1/ 50 years at 7% interest. Includes replacement costs and land.

2/ Does not include treatment.

FOREWORD

Authorization

On February 11, 1974, the Omaha District, U.S. Army Corps of Engineers authorized Harza Engineering Company to prepare a Wastewater management plan for alleviating the combined sewer overflow problem in Omaha.

Source of Data

This study is based on available references, including published reports and unpublished data collected from the City of Omaha and the Airport Authority.

Acknowledgments

Acknowledgement is given for the advice and assistance provided by the Omaha District, Corps of Engineers and the staff of the Omaha Department of Public Works.

Principal participants in Harza Engineering Company for this assignment were the following:

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REPORT

Chapter I

GENERAL BACKGROUND

Overflows from Omaha's combined sewer system are discharged to the Missouri River on the order of 50 times per year. These overflows of untreated sewage and storm runoff amount to an estimated five billion gallons per year or a yearly average of 12 million gallons per day. Although the impact of these overflows on the Missouri River has not been fully determined, this practice will be in violation of state regulations after December 31, 1975.

The abatement of pollution from combined sewer overflows is only one area of wastewater management planning that is now being undertaken in the Omaha area. Other water quality management planning activities involve an evaluation of the sewer systems in Omaha and Council Bluffs and the formulation and evaluation of alternative regional wastewater management plans. These current wastewater management studies for the Omaha area are directed toward meeting the objectives of the Federal Water Pollution Control Act Amendments of 1972 (PL 92-500).

The main objective of this study is to formulate and evaluate alternative measures for abatement of pollution of the Missouri River caused by combined sewer overflows from Omaha. As part of this study, special problems in the Carter Lake area and the impact of the proposed Riverfront Development Program are also assessed.

Setting

The study area, as outlined in Plate 1, consists of Carter Lake, Iowa, and the part of Omaha that drains directly to the Missouri River. This area, designated the Omaha-Missouri River sewerage system, covers more than 25,000 acres. The rest of Omaha is in the Papillion Creek watershed and drains eventually to the Missouri River via Papillion Creek. The types of land uses, population, topography, geology, and climate of the study area all have a bearing on the wastewater management planning.

Land Use

Land use within the study area includes industrial, commercial, residential, and vacant and parkland. The type of land use has considerable bearing on the performance of combined sewers because of the wide variation of storm runoff with different land uses. Land use within the service area of the sewerage system varies from highly impervious industrial areas, with runoff of up to 90 percent of rainfall, to very pervious areas such as parks, with as little as zero to 10 percent runoff.

Present land use within the study area, as shown in Table I-1, indicates that most of the land has been developed, either residentially or industrially. Land uses are fairly well distributed, however, so that the composite land use of large areas does not vary substantially throughout the study area.

Table I-1

PRESENT LAND USE IN THE STUDY AREA

Land Use	Area, Acres
Industrial-Commercial	4,220
Residential	13,480
Vacant-Parks	8,120
	25,820

Population

Recent population growth in Omaha has resulted in westward expansion of the city limits rather than an increase in population density in the study area. Future growth, however, is expected to occur in the study area as the Riverfront Development Program is carried out and the downtown central business district is redeveloped. This growth is projected to increase the study area's present population from 173,000 to 189,000 by 1995 and to 205,000 by 2020. A breakdown of the growth in the study by sewer service areas is presented in Appendix A.

Topography

Topography in the study area is greatly influenced by the Missouri River, which forms the eastern limit of Omaha-Missouri River sewerage system. Along the eastern edge of the study area, the land is low-lying flood plain, now protected by a levee. In the northern half of the study area, the land rises relatively gradually to an elevation 200 ft

above the Missouri River, five miles from the river. In the southern half the land rises sharply with steep bluffs 200 to 500 ft from the river. These bluffs are 100 to 150 ft above the river level.

The northern half of the study area contains a large flood plain area that is relatively flat. This area rises only 40 ft above the Missouri River. Carter Lake, an oxbow lake, lies in the center of this area and receives surface runoff from the surrounding 4000 acre area. The rest of the northern half contains rolling hills and valleys that slope toward the Missouri River.

The southern half of the study area also contains rolling hills and valleys. The main topographic difference between the two halves is the steep bluffs close to the river in the southern half.

Trunk sewers in the study area follow the natural drainage ways. Except for the flat area around Carter Lake, the study area drains directly to the Missouri River. Sewer gradients generally follow the natural slope of the land.

Geology

The study area lies within the Missouri River valley, which has been eroded into the broad loess-mantled upland till plain. The bluffs above the Missouri River are composed mostly of loess and glacial till; and in some cases bedrock is exposed along the bluffs. Overburden in the valley consists of alluvium and alluvial terraces. The alluvium ranges in thickness from 60 ft to 100 ft, and is composed of gravel, sand, silt, and areas of clay that are up to 40 ft in thickness.

Beneath the alluvium, about 2000 ft of sedimentary rock overlie Precambrian granite in the Omaha area. The rock consists largely of shale, sandstone, limestone, and dolomite ranging in age from Pennsylvanian to Cambrian. The uppermost bedrock unit underlying the Omaha area is the Kansas City Group, which consists of relatively pure limestone, though much of the Group is thin bedded. Underlying the Kansas City Group is the Marmaton Group, consisting of about 120 ft of shale and beds of limestone. Below the Marmaton Group is the Cherokee Group, consisting of about 300 ft of thin bedded shales, siltstone, and sandstone. This Group base marks the base of the Pennsylvania rocks. Below the Pennsylvanian rocks, and about 450 to 700 ft below the surface, is about 200 ft of massive limestone and dolomite of Mississippian age. A discussion of the engineering geology is presented in Appendix C.

Climate

Omaha is located between two distinct climatic zones, the humid east and the sub-humid west. As a result the summers typically are warm and the winters cold and dry. The study area is subject to periodic and rapid changes in weather, such as high-intensity thunderstorms that may center over a localized area. The mean annual precipitation is 28.5 inches and the average annual snowfall is 31.2 inches. Only 10 percent of the total annual precipitation falls during the winter months, while 75 percent of the total falls during the 6-month period from April to September.

Rainfall intensity-duration-frequency tables for the Omaha area are presented in Appendix A, Tables A-3 and A-4. These tables provide a basis for estimating the magnitude and intensity of rainfall events of different recurrence intervals. Data is presented for precipitation events of 1, 2, 5, 10, and 25-year recurrence intervals.

Overview of Problem

Drainage in about 22,000 acres of the study area flows naturally toward the Missouri River, a circumstance taken full advantage of during the early phases of constructing sewers in Omaha. Combined sewers to carry both sanitary and storm water by gravity were constructed near the turn of century and into the 1930's as Omaha grew. Domestic sewage, industrial waste, and storm water runoff were discharged without treatment to the river. This practice of continuously discharging raw sewage to the Missouri River ended in 1964 with the construction of a riverfront intercepting sewer system that conveys the sewage to a primary treatment plant. No changes to this system have been constructed since 1964 so that whenever the rate of storm water runoff exceeds the capacity of the intercepting sewer (about 50 times per year), raw sewage is discharged to the Missouri River.

Combined sewer overflows annually contribute an estimated four million pounds of BOD_5 and eight million pounds of suspended solids to the Missouri River. The

volume of overflows is estimated to be over 5 billion gallons per year. The water quality of the overflow is expected to vary considerably from event to event; but on the average, it is estimated to have concentrations of 100 mg/l BOD_5 and 250 mg/l suspended solids.

The solution to the special problems in the Carter Lake area and the impact of the Riverfront Development Program could affect the approach taken to alleviate the combined sewer overflow problem in Omaha. Changes in the sewage flow rate, in the required treatment plant capacity, in the storm runoff rate, in the peak combined sewer flow, and in the land use could result depending upon the impact of the Riverfront Development Program and the solution to the Carter Lake special problems. The magnitude of these changes could have some impact on the existing sewerage system, and, consequently, on the combined sewer overflow problem.

Chapter II

HIGH GROUND WATER AND SEPTIC TANK PROBLEMS IN THE CARTER LAKE AREA

Problems with septic tank system performance and high ground water are prevalent in the area surrounding Carter Lake. Solutions to these problems may affect sewage and runoff tributary to the Omaha-Missouri River Sewerage System. The affect on the sewerage system may, in turn, impact on the formulation and evaluation of alternatives to alleviate combined sewer overflow problems.

Description of the Carter Lake Area

The Carter Lake area is bounded on the north, east, and south by the Missouri River. The western edge of this area extends from the Missouri River north along the Grace Street outfall ditch, the North Omaha Interceptor, and 13th Street to the Missouri River. The focal point of this area is Carter Lake, an oxbow lake around which three communities and Eppley Airfield are located. Carter Lake, Iowa, is located in the crescent formed by Carter Lake. The community of East Omaha, now annexed to the City of Omaha, lies along the southeastern edge of Carter Lake. East of Carter Lake and north of East Omaha is Eppley Airfield, and due north of Carter Lake is a small residential area which is part of the City of Omaha (referred to as the North Area for purposes of this study). The locations of these areas are shown in Plate 2.

Population and Land Use

The total 1970 population of the Carter Lake area was about 6000 people of which over half reside in Carter Lake, Iowa. The distribution of the present and future population in the Carter Lake area is presented in Table II-1. Population is projected to increase only for Carter Lake, Iowa. All of the present and projected population is assumed to be within the three existing residential areas.

Table II-1

CARTER LAKE AREA POPULATION PROJECTIONS 1/

Area	Population 1970	1995	2020
Carter Lake, Iowa	3630	5445	7650
North Area	1100	1000	1000
East Omaha	1200	1030	1030
Total	5930	7475	9680

1/ Source: Metropolitan Utilities District
See Appendix A for alternative population projections.

Present land use is either residential, vacant, or industrial. Approximately 500 acres of vacant land north and northwest of the North Area are planned for new industry. Areas to the North and East around Carter Lake could become industrial within the proposed planning period. As part of the Riverfront Development Program, new low density residential development in Carter Lake, Iowa, and expansion of Eppley Airfield to the south is planned. Except for Carter Lake, Iowa, no further residential development is projected for the Carter Lake area, as indicated in Table II-1.

Topography and Geology

The Carter Lake area lies within the flood plain of the Missouri River. It is a generally flat, low-lying area, protected from Missouri River floods by a levee. Elevations within the Carter Lake area generally range from 970 to 980 ft above mean sea level (msl). The Missouri River stage north of the area is about 970 ft msl during the navigation season.

The geology of the Carter Lake area is similar to other flood plain areas in the Omaha vicinity. It is underlain with 60 to 100 ft of flood-plain alluvium, which is reported to be fine to medium sand and fine pebbles, covered by a mantle of silt and areas of clay ranging from about 4 ft to 20 ft thick.

Soil survey maps provided by the Soil Conservation Service (SCS) have delineated predominately tight surficial soils over the North Area. The SCS has mapped the North Area as Albaton silt loam or silty clay and has classified both soils as unsuitable for septic tank and tile field disposal of sewage because of the generally low permeability and periodically high water table conditions. The East Omaha area is mapped as fill (man-placed) material. It is presumed to have wide-ranging soil properties dependent upon the origin of fill material at any given one location. The Carter Lake, Iowa, area is mapped as deep, well drained soils in sandy and loamy alluvium.

Carter Lake

Carter Lake receives surface runoff from the airport and from a residential area northwest of the lake. Water levels in the lake are maintained below 970 ft msl by pumping lake water to the Omaha sewerage system. Carter Lake is effectively used as a stormwater runoff retention basin. This use, however, does not adversely affect septic tank system performance nor ground-water levels in the Carter Lake area.

On the belief that high lake levels caused softening of the soil under the airport runways, 971.5 ft msl was established as the upper limit desired for Carter Lake. The City of Omaha tries to hold the elevation at 970 ft msl to provide emergency capacity in the lake to absorb storm runoff without raising the lake level above 971.5 ft msl.[1/] The effect of stabilizing Carter Lake water levels on the airport runways has been found to be negligible and it is evident that the Missouri River stage has a much greater influence on the ground water level in the Carter Lake area. Thus, the concept of lowering the ground water to improve septic tank operation by stabilizing Carter Lake water levels is not technically feasible.

Ground Water

The unconsolidated aquifer is relatively well connected to the deeply incised Missouri River Channel. Thus, it tends to receive flow at the northerly side of the area and

1/ Review Report, Carter Lake, U.S. Army Corps of Engineers, November 1960.

to discharge to the southerly, or downstream, side. An additional and greater amount of recharge is added by infiltration from rainfall over the area. Excessive landward seepage during high river stages is prevented by relief wells that are distributed along the landward side of the levee and flow to drainage ditches at about elevation 972 to 975 ft msl. Water levels above those of the unconsolidated aquifer occur during periods of high rainfall where surface soils have low vertical permeability and permit a perched water table condition to develop.

Water levels at the Eppley Airfield piezometer ranged from 969.5 to 972 ft msl during most of 1972. Piezometer readings above 975 for locally heavy rainfall periods during late March and early April and up to 972.5 ft msl during September were recorded. Similar levels are reported in a 1960 U.S. Army Corps of Engineers study[1/] that shows a general gradient of some five to ten feet from north to south across the area.

Limited water-level records[1/] [2/] indicate that Carter Lake comprises a net discharge area most of the time by pumping water out of the lake when the water level exceeds 970 ft msl, or by evaporation during dry months. The operating experience on pumping to maintain lake levels and the clay thickness shown[1/] in lake-bottom borings suggest that Carter Lake is poorly connected to the groundwater aquifer.

1/ Ibid.

2/ Unpublished data on river, lake, and piezometer water levels, Omaha Airport Authority, 1973-74.

Problem Identification

Residents of East Omaha and the North Area use septic tank and tile field wastewater disposal systems. The City of Omaha has extended sewerage service to Carter Lake, Iowa, Eppley Airfield, and several homes in East Omaha. The locations of the existing wastewater disposal systems are shown in Plate 3.

As indicated by a survey of the residences in the East Omaha and North Area by the Douglas County Sanitarian, many of the septic tanks are undersized, the systems are 20 to 50 years old, and it is doubtful whether many of them have tile fields. Most of these systems were installed without having a percolation test made to determine the capability of the soil for a tile field. About half of the residents using septic tanks in this area have reported various degrees of difficulty with these septic tank systems during 1973.

The high ground-water problem is not as well documented as the reported septic tank problems. The normal fluctuation of water levels does not affect existing structures at the airfield where the land surface is mostly above elevation 975 ft msl.[3/] It does, however, cause infiltration to airport storm sewers, and occasionally has caused sufficient loss of fines to cause collapse of the storm sewer. The County Sanitarian has reported that residents in the North Area and East Omaha have a problem with flooded crawl spaces, apparently

3/ Wirth, Milton, Airport Authority, Personal Communication.

caused by high ground water.[4] No problems with high water tables are reported for the Village of Carter Lake, Iowa.

Recommended Plan

The provision of a separate sanitary sewer system to convey wastewater from the East Omaha and North Area to the Omaha-Missouri River sewage treatment plant is recommended for elimination of the problems associated with poor septic tank performance. Measures to alleviate the present problems caused by the high ground-water levels are not recommended because of their technical infeasibility. In addition, any residual high water table problem will be small if the septic tank problems are eliminated. It is, however, recommended that future construction in the Carter Lake area incorporate appropriate provisions in recognition of the presence of a high ground-water level.

The areas for which provision of sanitary sewers is recommended are shown in Plate 4. The North Area would be served by a trunk sewer, estimated to be 12-inches in diameter and 8000 ft long, leading to the North Omaha Interceptor, which conveys sewage to the sewage treatment plant. The East Omaha area would be served by an existing trunk sewer that extends to Eppley Airfield. Part of the trunk sewer may have to be paralleled in future years if expansion

4/ Bosen, Jack, Douglas County Sanitarian, Personal Communication.

of the airport facilities continues. The initial investment cost of the collection system and treatment plant addition is estimated to be $1,350,000 and the annual operation and maintenance costs are estimated to be $13,000.

Industrial development in the Carter Lake area may stimulate construction of trunk sewers to the North Area and to East Omaha. In this case, connecting the North Area to the Omaha-Missouri River Sewerage System would be more cost-effective. However, the cost of providing sewer service to the residences of the North Area and East Omaha is mainly attributable to house sewers and laterals. Consequently, the present plans of Riverfront Development Program will not increase the feasibility of providing sewer service to East Omaha and the North Area.

Impact on the Omaha-Missouri River Sewerage System

The additional sewage flow to the Omaha-Missouri River Sewerage System caused by extending sewers to East Omaha and the North Area is estimated to be 0.23 mgd, average daily flow. In addition, this may be increased by significant infiltration caused by the high ground-water levels. The additional BOD_5 and suspended solids are estimated to be 385 and 469 pounds/day, respectively. These amounts represent 0.6 percent of the present average daily flow, 0.2 percent of the BOD_5 load, and 0.3 percent of the present suspended solids load received at the treatment plant. Therefore, extension of sewer service to these areas will have negligible impact on the existing sewerage system.

Because of the flatness of the area to be served around Carter Lake, provision of sewage lift stations may be necessary. These lift stations will add to the operation and maintenance tasks of the city crew that services the collection system. To minimize infiltration, new sewers should have tight joints, adequate bedding and proper alignment. Adequate inspection of the installation of the sewers is also necessary to reduce the potential for infiltration.

Alternatives

Several alternatives were formulated and evaluated prior to selection of the recommended plan as described in the previous section. These conceptual alternatives consisted of the following:

1. Lower the groundwater table by
   a) Constructing groundwater drains, or
   b) Allowing infiltration to sanitary sewers.
2. Replace all malfunctioning septic tank systems with
   a) New, larger septic tank systems, or
   b) Home unit package treatment plants
3. Convert all malfunctioning septic tanks to holding tanks and have City of Omaha pump out tanks and transport wastes to treatment plant.
4. Relocate residents of East Omaha and the North Area.

Alternative No. 1

Construction of groundwater drains or allowance of infiltration to sanitary sewers is technically and economically impractical because of the tight surficial soils and their low permeability in the North Area and East Omaha.

Alternative No. 2

Alleviation of the septic tank performance problems by the replacement of poorly operating septic tank systems with another system that relies on efficient use of a drain field is technically unsound. Both new septic tank systems and home treatment units require an adequate drain field for effluent disposal. Soils in the North Area are classified as poorly drained, tight surficial soils and unsuitable for septic tank installation. In addition, in the North Area the ground water is too high. In the East Omaha area, the drain fields may be in adequate soils, but the ground water rises too high for adequate year-round operation of systems at elevation 974 ft msl or below. Residences above 974 ft msl and located on adequate soil for a septic tank drain field in East Omaha could replace an inadequate septic tank system with a new, similar type system; but this alternative is applicable only to small areas.

Alternative No. 3

The conversion of all malfunctioning septic tanks to holding tanks and subsequent hauling of sanitary wastes to

the sewerage system is an extremely expensive and cumbersome alternative. The high cost for operation and maintenance is one of the disadvantages. Just as importantly, it is expected that most of the septic tanks in the older systems (greater than 10 years old) do not have enough capacity to hold more than three days waste from a residence. With about 500 residences in the problem areas, this would mean that about 170 residences per day must be tended or new tanks installed. Because of the high costs involved and the implementation and operation problems anticipated, this alternative was not recommended.

Alternative No. 4

Relocation of the residences that have septic tank problems would end the problems of septic tank systems in the Carter Lake area. However, in addition to the high cost (on the order of 10 times to cost of the recommended plan), the disruptive effects would be high, and public acceptance is unlikely. However, if these areas become industrial the objections would be somewhat reduced.

Chapter III

ASSESSMENT OF THE IMPACT OF RIVERFRONT DEVELOPMENT ON THE OMAHA - MISSOURI RIVER SEWERAGE SYSTEM

The Riverfront Development Program (RDP) is a comprehensive program for integrated development of the area adjacent to the Missouri River in Mills, Pottawattami, and Harrison Counties in Iowa and Sarpy, Douglas, and Washington Counties in Nebraska. The program undertaken by the Riverfront Development Authority consists of projects to encourage new growth along both sides of the Missouri River for about 53 miles. Projects that are to be undertaken in the study area consist of five new industrial site developments, six new residential developments, one commercial shopping area, two commercial marinas, and conversion of more than 100 acres of developed commercial-industrial land to open space and parks. The locations of the RDP projects are presented in Table III-1. Residential developments, termed "New Towns-in-Town", are also part of the RDP. Although their areal extent is undefined, the general locations of possible development are shown in Plate 5.

The impact of the RDP on existing sewerage system affects the formulation and evaluation of alternatives to alleviate the combined sewer overflow problem. Implementation of the projects in the Omaha area may change the rate and volume of storm runoff tributary to the sewerage system and may also change the characteristics of the dry-weather flow. These changes could affect not only the size of the alternative

facilities, but also their possible locations. The land use planned for certain areas under the RDP may be in conflict with some of the alternatives or may contribute to alternatives such as sewer separation and storm runoff retention.

Table III-1

LIST OF PROPOSED
RIVERFRONT DEVELOPMENT PROJECTS

Location Number 1/	Present Land Use	Proposed Project	Area, Acres
1	Open Space	Commercial Marina	50
2	Open Space	Medium Density Residential	80
3	Open Space	Light Industrial	240
4	Open Space	Light Industrial	320
5	Industrial	Medium Density Residential	60
6	Open Space	Low Density Residential	175
7	Open Space	Low Density Residential	100
8	Open Space	Parks and Recreation	660
9	Residential	Commercial, Shopping Area	35
10	Open Space	Light Industrial	40
11	Open Space	Open Space	95
12	Open Space	Light Industrial	40
13	Open Space	Commercial Marina	90
14	Commercial-Industrial	High Density Residential & Park	180
15	Open Space	Industrial	100
		Total	2265

1/ Number corresponds to location number in Plate 5.

Assessment of Impact

Implementation of the RDP will change land use of about 2265 acres in the study area. A change in the land use of an area is often accompanied by a change in the drainage characteristics and a change in the amount and strength of wastewater generated within that area. These changes have an impact on the sewerage system. The magnitude of that impact is dependent upon the area of land involved, the difference in present and new land use, and the difference in present and future activity within the area. The term "activity" is used to denote changes in the intensity of land use, such as changing a warehouse area to a meat-packing area but still retaining the land in an industrial land use category.

The impact of a change in land use on the sewerage system is identifiable in terms of dry-weather flows and wet-weather flows. Dry-weather flows are greatly influenced by the land use activity. Wet-weather flows are primarily influenced by the drainage characteristics.

Dry-Weather Flows

Dry-weather flow is wastewater without dilution by surface runoff and consists of domestic wastewater, industrial wastewater, and/or wastewater from commercial areas. The strengths and flow rates of the dry-weather wastewaters from each area can affect the operation of the collection system and the treatment plant. Table III-2 presents the estimated

range of change in the average dry-weather wastewater flows from each planned land use or activity change under the RDP. Peak flows discharged from each area might be as high as three times the average flow.

Sewer system. The existing sewer collection system in the Omaha-Missouri River sewerage system is a combined sewer system that receives surface water runoff and dry-weather sewage. Flows are conveyed to the treatment plant by an interceptor system as shown in Plate 6. The collection system, which drains eastward to this interceptor, is a combined system, but the single trunk sewer that serves Eppley Airfield and Carter Lake, Iowa, is a separate sanitary sewer.

The projected incremental dry-weather sewage flow tributary to the interceptor system varies greatly depending upon the type and size of industry that may locate in an area. The incremental sewage flow from a change in residential and commercial land use is, however, predictable within a much narrower range of flows. Consequently, the impact of the RDP can be assessed by evaluating the capacity of the interceptor system remaining for industrial flows after the commercial and residential flows have been accounted for. This capacity, which is estimated to be available for industrial flows, can then be compared to the range of industrial flows projected for an area.

The capacity available for future RDP industrial flow, as shown in Table III-3, is dependent upon the construction of a facility to allow treatment of combined sewer overflow.

Table III-2

ESTIMATED AVERAGE DRY-WEATHER WASTEWATER FLOWS FROM RIVERFRONT DEVELOPMENT PROJECTS IN OMAHA, NEBRASKA

Location Number	Wastewater Flow, mgd							
	Residential 1/	Commercial 2/	Industrial			Total		
			Low 3/	Medium 4/	High 5/	Low	Medium	High
1	-	6/	-	-	-	0.0	0.0	0.0
2	0.08	-	-	-	-	0.08	0.08	0.08
3	-	-	0.24	1.80	3.6	0.24	1.80	3.6
4	-	-	0.32	2.40	4.8	0.32	2.40	4.8
5	0.06	-	-	-	-	0.06	0.06	0.06
6	0.09	-	-	-	-	0.09	0.09	0.09
7	0.05	-	-	-	-	0.05	0.05	0.05
8	-	-	-	-	-	0.0	0.0	0.0
9	0.035	0.035	-	-	-	0.07	0.07	0.07
10	-	-	0.20	1.50	3.0	0.20	1.50	3.0
11	(-0.095)	-	-	-	-	(-0.095)	(-0.095)	(-0.095)
12	-	-	0.04	0.30	0.60	0.04	0.30	0.6
13	-	6/	-	-	-	0.0	0.0	0.0
14	0.32	-	-	-	-	0.32	0.32	0.32
15	-	-	0.10	0.75	1.5	0.10	0.75	1.5
Total						1.47	7.29	14.07

1/ 100 gpcd
2/ 1000 gpd/acre
3/ Low: 1000 gpd/acre of industrial land (Dry industry such as warehouse)
4/ Medium: 7500 gpd/acre of industrial land (Wet industry such as food processing)
5/ High: 15,000 gpd/acre of industrial land (Very wet industry such as papermill)
6/ Negligible

If no such facility is constructed, the total average dry-weather flow in the north part of the interceptor system must be no more than one-fifth of the sewer capacity. For the south part of the interceptor system, this fraction is one third.1/ These fractions were established by the U.S. Public Health Service, Iowa and Nebraska State Health Departments, and the City of Omaha. This constraint severely limits the future industrial growth in some areas. If an overflow prevention facility is constructed, then these dilution requirements will no longer apply and the available industrial capacity will be equal to the capacity of the interceptor minus the peak dry-weather residential and commercial flow.

The dilution requirement imposes a severe restriction on future industrial development in the Carter Lake area. Either industrial development will usurp interceptor capacity that is reserved for future residential growth, or only industries that generate relatively small amounts of wastewater can be allowed to develop if they are to be served by the Omaha-Missouri River sewerage system. This restriction imposed by the dilution ratio does not, however, preclude development of large water-using industries that would treat their own wastewater.2/

Treatment system. The present treatment plant for the Omaha-Missouri River sewerage system is a primary treatment

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1/ Interceptor Sewers and Sewage Disposal, Volume 1, Henningson, Durham, and Richardson, 1958.

2/ The practice of new industry providing treatment of its own wastewater is discouraged by the U.S. EPA, which stresses non-proliferation of treatment plants.

Table III-3

AVAILABLE CAPACITY IN NORTH-SOUTH INTERCEPTOR
TO CARRY FUTURE INDUSTRIAL FLOWS FROM RIVERFRONT INDUSTRIAL DEVELOPMENTS

Location on Interceptor System	Reference Numbers of RDP Projects Located Upstream of Interceptor Point 1/	Dry-Weather Capacity of Interceptor mgd	Amount of Incremental Industrial Flow That Can Enter Interceptor From RDP Projects, mgd	
			Average Flow 2/	Peak Flow 3/
Mormon Street	-	2.7	.8	6.9
Minne Lusa Street	-	11.5	.8	11.8
Cornish Blvd.	2,3,4,5	51.0	.8	11.8
Grace Street	2,3,4,5,6,7,9,10,12	14.2	.8	11.8
Burt-Izard Streets	2,3,4,5,6,7,9,10,12,14	17.4	.8	11.8
Leavenworth Street	2,3,4,5,6,7,9,10,12,14	31.8	5.4	29.4
Missouri Avenue	2,3,4,5,6,7,9,10,12,14	36.0	6.5	34.0

1/ Reference numbers correspond to numbers in Table III-1, Table III-2, and Plate 5.

2/ If overflows of combined sewage continue, the total average flow must be less than a third or one fifth of the interceptor system capacity, depending on location.

3/ If overflows of combined sewage are eliminated, allowable peak dry weather flow equals sewer capacity.

plant, sized for 72 mgd average dry-weather flow. This plant was designed for a raw waste load of 270,000 pounds of BOD_5 per day, 244,000 pounds of suspended solids per day, and 15,000 pounds of grease per day. The present average dry weather flow is estimated by the City to be 28 mgd.

The increase in average dry-weather wastewater flow from residential growth in the entire Omaha-Missouri River sewerage area is estimated to be 1.5 mgd by 1995, based on 100 gpcd. The incremental increase in wastewater flow as a result of the RDP projects is estimated to range from 1.5 to 14.0 mgd. Thus, the 1995 estimated average dry-weather flow to the treatment plant could range from 41 mgd to 53.5 mgd. This range of flows is well below the present design capacity of the treatment plant.

The present primary treatment plant is estimated to receive 180,400 pounds of BOD_5 per day and 163,500 pounds of suspended solids per day. Additional loads from projected population growth are estimated to be 3000 pounds of BOD_5 per day and 3600 pounds of suspended solids per day by 1995. By 1995, the increase in BOD_5 and suspended solids loading due to changes in land use could be 86,600 and 76,900 pounds per day, respectively, without overloading the treatment plant. The 86,600 pounds of BOD_5 in 14 mgd is equivalent to a concentration of 742 mg/l, which is about twice the strength of the average industrial waste tributary to the treatment plant. Thus, it appears that implementation of the RDP projects would not cause an organic overload on the existing treatment plant.

There is one main impact on the treatment plant that would result if the RDP is implemented. The increase in flow, BOD_5, and suspended solids associated with the RDP will increase the operation and maintenance costs of the existing treatment plant. In addition to the additional pumping and other power requirements, more sludge will be generated and disposed of. This impact cannot be assessed quantitatively at this stage in the RDP program, but it is not expected to be any greater than would be expected if the developments took place elsewhere within the sewerage system.

Alternatives for Abatement of Combined Sewer Overflows. Several of the alternatives for abatement of combined sewer pollution require surface storage sites to be located in areas that are RDP sites for new industrial development. If the RDP projects in areas No. 3, 4, 10, and 12 were committed as planned, the nonavailability of land in that vicinity poses a severe constraint on alternatives that require surface storage in the area around Carter Lake.

Wet-Weather Flows

Wet-weather flows in a combined sewer consist of a dry-weather flow plus surface runoff and inflow/infiltration. The peak flow rate during wet-weather conditions is primarily a function of the rate of surface runoff, which, in turn, is a function of land use and rainfall intensity. Land use is important because it influences the amount of land that is impervious to rainfall. Heavily industrialized areas may

have 95 percent of the total area impervious to rainfall. Open space areas, such as parks, may have 95 percent of the area pervious to rainfall. The runoff coefficient, C, in the rational formula, indicates the imperviousness of an area. High values of C correspond to high percentages of impervious land.

The impact of the RDP projects on the peak wet weather flows can be assessed by examination of the change in the values of C in each sewer service area. Values of C before and after the realization of the RDP for each sewer service area are presented in Table III-4.

As indicated in Table III-4, the RDP projects will have negligible impact on the volume of runoff from the combined sewer service areas, but will significantly increase the runoff in the Carter Lake area. This area, however, is unsewered, and consequently, there is no impact on the existing sewerage system.[1/] If the area is sewered in the future, the system will not be a combined sewer system so that storm runoff will not enter the existing collection system from this area.

It is possible that in a few local areas the RDP development may cause the runoff to occur faster than before. These

1/ There will be some impact on the existing sewerage system if the City continues to pump Carter Lake into the North Omaha Interceptor. This impact would be minimal because the pumping would occur after the storm event. Any pumping during a storm event will be negligible relative to the amount of overflow already occuring. Although the population is expected to increase with the RDP projects a slight decrease in the runoff coefficient was assumed for the remaining areas due to an anticipated increase in open space and park areas.

situations may cause local flooding problems; but with adequate provisions for control of runoff included in the site development, flooding conditions can be avoided. Such provisions may include parking lot detention, roof top storage, and porous pavement. The City of Omaha may require controlled release of surface runoff from future developments to ensure that new developments do not cause local flooding.

Table III-4

RUNOFF COEFFICIENT VALUES FOR
SEWER SERVICE AREAS AS AFFECTED
BY THE RIVERFRONT DEVELOPMENT PROGRAM

	Runoff Coefficient 1/		Runoff Volume, 1/ A-ft	
Service Area	Present	Future	Present	Future
Grace Street	0.69	0.68	480	470
Burt-Izard Street	0.78	0.76	730	710
Leavenworth Street	0.73	0.71	620	605
Monroe Street	0.65	0.65	470	470
Carter Lake Area 2/	0.32	0.46	565	810

1/ Recurrence interval of 25 years.
2/ Area east of interceptor system, in the vicinity of Carter Lake.

Guidelines for Future Assessment

As the RDP advances from the various stages of planning into the design stages, the impact of the proposed projects will be assessable in more quantitative terms. The generalized nature of the RDP plans now precludes a detailed assessment

of a change in land use on the sewerage system. Guidelines for assessment of future land use changes are presented below as general evaluation criteria.

Type of Land Use

The type of land use affects both the dry-weather and the wet-weather flows discharged from an area. Dry-weather flows are primarily a function of activity, which, in turn, is dependent upon land use. Wet-weather flows are dependent upon land use because they are a function of the percent of impervious area.

Once land use is established, the peak runoff rates and total runoff volumes for various recurrence interval events can be determined and checked against the hydraulic capacity of the receiving sewers. Problems of surcharging the sewer system and causing flooding may be avoided by limiting the runoff discharge rate from an area tributary to the combined sewer system. Peak flows can be attenuated by proper design of surface drainage for an area under development.

Runoff volumes from an area may affect the storage component of an alternative to abate combined sewer pollution. As the amount of imperviousness increases, the runoff volume increases. Unless the land use change involves an area greater than 100 acres, the impact will probably be negligible.

Change in Intensity of Use

A change in intensity of land use will probably result in the greatest impact to the sewerage system. The following

criteria are presented for the assessment of the impact when a change in the intensity of land use is planned.

Location. The location of a new housing development, industrial site, or commercial area is important in the assessment of the new activities impact on the sewerage system. The location identifies which sewers will serve the area and establishes the physical constraints on the development.

Wastewater Flows. The wastewater flow generated from a new activity depends upon the type of activity and its size. Wastewater from industrial plants can often be estimated on the basis of gallons per unit of product. This basis varies from industry to industry and is helpful only when the industry type is known. Wastewater from residential areas can be estimated on the basis of 100 gpcd for average daily flow.

The impact of the flow from a new land activity must be assessed for the collection system and the treatment plant. The collection system may be locally overloaded by a large point source of flow from a new activity.

For the assessment of the impact on the collection system, peak flows can be estimated fairly accurately for residential, commercial, and open space areas. Industrial peak flows, however, vary not only from industry to industry, but also from plant to plant, shift to shift, etc. The impact on

the collection system can be minimized by setting a maximum allowable peak flow from an area so that the sewer capacity is not exceeded.

Changes in flow to the treatment plant affect the power and chemical requirements, the detention times, and the overflow rates and can, thus, significantly affect the process efficiency of the treatment plant. The magnitude of the effect on the plant can be assessed by comparing the change in flow to the existing average daily flow. As the magnitude of change increases, the effect on the plant increases.

Wastewater Characteristics. The characteristics of the wastewater affect the efficiency of the treatment plant and the solids handling units. As the BOD_5 loading increases, the efficiency of the plant can decrease. Additional BOD_5 and suspended solids add to the amount of solids to be handled and to be disposed of. These requirements will affect operation and maintenance costs, but no more than if the development cccurred elsewhere in the service area.

In addition to BOD_5 and suspended solids, there are other wastewater characteristics that can affect the treatment plant and/or the collection system. Certain wastes should be excluded from the sewers. These wastes include wastes that create a fire or explosion hazard in sewers or treatment plant; wastes that impair the hydraulic capacity of the sewers; and wastes that, in any quantity, create a hazard to people, to the sewer system, to the treatment process, or to the receiving waters.

Examples of wastes in these three categories include: 1) gasoline, fuel oil, and cleaning solvents; 2) ashes, sand, cinders, metal, rags, and unshredded garbage; and 3) hydrogen sulfide, sulfur dioxide, and cyanides at dangerous levels.

Other characteristics that have an adverse effect on the sewerage system include temperature (above $150^{\circ}F$), volatile oils, strongly acidic wastes, highly alkaline wastes, and toxic concentrations of copper, chromium, lead, zinc, arsenic, and nickel. These characteristics are usually found only in industrial wastes and can be prevented from entering the sewerage system if proper measures are taken.

It should be noted that the location and flow rates are important when assessing the impact of the waste strength and characteristics. The impact of a single point discharge is lessened as it is diluted in the collection system. Consequently, the farther north a point source is located in the Omaha-Missouri sewerage area, the more attenuated its effect will be at the treatment plant. However, for wastewaters with characteristics that affect the collection system, problems may develop at and near the point of discharge.

Time Land Use or Land Activity Change Occurs

The scheduling of a change in land use or activity has an impact on the planning of new treatment facilities and sewers. Future additions to a sewerage system are not only

dependent upon the wastewater flows and loadings from an area, but also when those changes are going to occur.

The assessment of the impact of proposed changes in land use and the activity on the sewerage system can be simplified by placing constraints on future development to ensure that the sewerage system will accept the change in loading. Constraints include land use zoning, pretreatment requirements for certain waste discharges, regulations on peak flow rates, and sewer ordinances. If changes in land use and activity are planned for, any adverse impact on the sewerage system can be minimized. Planning may involve coordinating construction of a new RDP project with the already needed construction of a new sewer or with part of a sewer rehabilitation program.

Chapter IV

DESCRIPTION OF THE COMBINED SEWER SYSTEM

Mixed storm runoff and sanitary sewage can be discharged to the Missouri River from 20 overflow points in the Omaha-Missouri River sewerage system. Overflows occur whenever the rate of storm runoff and sanitary sewerage flow exceeds the capacity of the interceptor system. These discharges of untreated combined sewage will be in violation of Nebraska state law after December 31, 1975. In order to formulate aternative plans to prevent the combined sewer overflows, an understanding of the sewerage system and its operation is necessary.

Sewerage System

Sewage in the study area is conveyed by trunk sewers in each service area to an interceptor system that, in turn, conveys the sewage to a treatment plant providing primary treatment. This system, as shown in Plate 6, also drains storm runoff from service areas east of the interceptor system. Only one service area located east of the interceptor system has separate storm and sanitary sewers. This area is the Carter Lake-Eppley Airfield-East Omaha service area.

The existing collection system was initiated near the turn of the century. Many of the trunk sewers, built between

1900 and 1930, are brick construction and in need of repair. These trunk sewers originally conveyed sanitary sewage and storm runoff directly to the Missouri River. In 1964 an interceptor system was constructed to divert sanitary sewage and some of the storm runoff to a primary treatment plant. No other major changes to the sewerage system have been constructed so that discharges of combined sewage now occur about 50 times per year.

The Omaha-Missouri River sewerage system consists of ten main service areas, eight of which are combined sewer service areas and two are essentially separated sanitary and storm sewer service areas. Wastewater from these service areas can overflow from any one or more of 22 overflow points. Of these overflow points, 19 are combined sewer overflow points, two are sanitary sewer outlets serving the Carter Lake area, and the remaining point is for separate storm sewer overflow in the Bridge Street area.

Service Areas

The sewer service areas tend to conform to the natural drainage boundaries. In several of the service areas, the main trunk sewers were constructed in natural drainage ways such as a creek or stream bed. Each service area consists of a sewer collection system that conveys sewage and, in nine of the areas, storm water runoff via one or more trunk sewers to the interceptor system. Only one service area, the Carter Lake-Eppley Airfield-East Omaha area, does not have combined sewers.

The amount of impervious land within a service area directly affects the volume and rate of storm runoff. The runoff coefficient, used in the rational formula, is one measure of the amount of impervious area within a service area. As shown in Table IV-1, the runoff coefficients vary between 0.40 in the Mormon Street area to 0.78 in the more built-up Burt-Izard service area.

The Carter Lake-Eppley Airfield-East Omaha service area has no effect on the combined sewer overflow problem because no storm runoff from this area enters the interceptor system. The emergency by-passes to the Missouri River from Eppley Airfield and Carter Lake, Iowa, are retained only for use in case of mechanical failures. The Bridge Street service area has been recently separated. Only one sanitary sewer connection to the storm sewer remains so that, for planning purposes, the area is considered totally separated.

Table IV-1

SEWER SERVICE AREA CHARACTERISTICS

Sewer Service Area	Drainage Area, Acres	Runoff Coefficient 1/	Number of Combined Sewer Overflow Points in Service Area
Bridge Street	3915	2/	0
Mormon Street	315	0.40	1
Minne Lusa St.	5360	0.61	1
Grace Street	1680	0.69	1
Carter Lake-Eppley Airfield-East Omaha	4730 2/	2/	0
Burt-Izard St.	2270	0.78	2
Leavenworth St.	2060	0.73	4
Greater Missouri	1620	0.50	7
South Omaha	2115	0.59	2 3/
Monroe Street	1760	0.65	1
Total	25,825	0.64 wt. avg.	19

1/ Based on 25 year recurrence interval and without RDP being Implemented.
2/ No storm drainage enters system.
3/ Includes "U" Street outlet.

Source: U.S. Army Corps of Engineers.

Interceptor System

The interceptor system consists of the North Omaha Interceptor, the South Omaha Interceptor, grit removal facilities and pump stations, and diversion structures. It conveys wastewater to the treatment plant from all main service areas except two, the South Omaha and Monroe Street areas. Trunk sewers from these areas discharge directly to the sewage treatment plant. The interceptor system was designed to convey all wastewater to the Missouri River plant so long as the proportion of the storm water flow to the design dry-weather flow does not exceed a prescribed ratio. The ratio varies depending upon the location of the overflow point. For the North Omaha Interceptor, which is upstream of the Council Bluffs raw water intake, the ratio is 5 to 1. For the South Omaha Interceptor, the ratio is 3 to 1. Flows causing this ratio to be exceeded are not diverted to the interceptor system and are discharged without treatment to the Missouri River.

North Omaha Interceptor. The North Omaha Interceptor collects the sanitary sewage from the Bridge, Mormon, Minne Lusa, Grace, and Carter Lake-Eppley Airfield-East Omaha service areas. The sewer ranges in size from 30 inches in diameter near Bridge Street to a 10 ft by 7.5 ft box sewer at Grace Street. There are no pumping stations in the North Omaha Interceptor. During times of storm, however, the portion of sewer between Minne Lusa Street and Grace Street surcharges.

South Omaha Interceptor. The South Omaha Interceptor connects with the North Omaha Interceptor at Grace Street. This interceptor collects flow from the North Omaha Interceptor and the Burt-Izard, Leavenworth, Missouri, and South Omaha service areas. Between its beginning at Grace Street and the Burt-Izard overflow, the South Omaha Interceptor consists of a 60-inch diameter gravity sewer. From the Burt-Izard lift station to the Leavenworth lift station, the interceptor consists of a 48-inch diameter force main; and from Leavenworth to the treatment plant, a 66-inch diameter force main.

Grit Removal Facilities and Pump Stations. Grit is removed from all wastewater pumped to the South Omaha Interceptor. During periods of high runoff, large, heavy objects are carried by the wastewater to the outfall. Some of these objects, such as bricks from the sewer lining and tree branches, are diverted to the grit removal and pumping facilities where they damage equipment. After the storm runoff stops, the facilities can be repaired, but only by by-passing the dry-weather sewage directly to the Missouri River.

All of the existing grit removal and pumping facilities along the South Omaha Interceptor lack provisions for preventing dry-weather overflow during periods of repair and mechanical failure. The facilities at Burt-Izard and Leavenworth have caused the greatest problems. The Burt-Izard facility is extremely critical because the flows from

both the North Omaha Interceptor and the Burt-Izard service area are pumped by this facility. Whenever repairs are necessary to this pumping facility, flows must be by-passed directly to the Missouri River. By-passing because of mechanical failures occurs frequently along the interceptor system.

The city is planning to add new grit removal facilities near the Leavenworth and Burt-Izard facilities. The new facilities would allow repair of the two existing facilities without causing a by-pass of dry-weather flow to the River.

Diversion Structures. The diversion structures, located near each main trunk sewer outfall, control the quantity of flow diverted to the interceptors. These structures direct up to three or five times the design dry-weather sewage flow through a coarse bar screen and into a diversion pipe that leads to the interceptor. Overflows occur whenever the runoff rate[1/] exceeds the capacity of interceptor system.

Sewage Treatment Facilities

The present waste treatment plant was placed into operation in 1964 and provides primary treatment before discharge to the Missouri River. The hydraulic design capacity of the primary treatment is 72 mgd. The plant was designed for a raw waste load of 270,000 pounds of BOD_5 per day and 244,000 pounds of suspended solids per day.

1/ Frequency and amount of overflow is discussed in Appendix A, Hydrologic Criteria

Because of the lack of adequate grit removal equipment in the interceptor system, not all of the dry-weather flow reaches the treatment plant. If the total flow actually reached the plant, it is estimated that the average dry-weather flow at the present time would be 28 mgd.[2/] More than 30 percent of this flow consists of industrial wastes with widely varying characteristics. Most of the industrial flow and load comes from the meat packing industry in the Monroe Street service area.

The existing plant is incapable of providing a sufficient degree of treatment for the combined sewer overflow. However, the city plans to upgrade this plant to secondary level treatment by provision of a completely mixed activated sludge process after the primary facilities. Secondary treatment must be added to this plant by December 31, 1975, in order to comply with Nebraska effluent standards. The treatment capacity of the planned secondary facility is 65 mgd.

Sewerage System Performance

The present sewerage system is designed to capture and convey all dry-weather flow to the existing primary treatment

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2/ In 1971 the sewage flow was estimated to 38 mgd, but some of the meat packing industry has left and the population has decreased.

3/ Data obtained from the City of Omaha, Public Works Department.

plant. Mechanical failures along the South Omaha Interceptor have, however, caused by-passing of raw sewage directly to the Missouri River. The City of Omaha[3/] estimates that the dry-weather by-passes totalled at least 4.5 billion gallons (13,000 A-ft annual or average of 12 mgd) during 1973. This amounted to an estimated[3/] 5400 tons of BOD_5 and 3700 tons of suspended solids.

The combined sewer collection system is also designed to carry sewage and storm flow to the interceptor system. In several areas the collection system becomes frequently overloaded during a runoff event, and basement flooding occurs. Along the interceptor system, there are 20 overflow points that are designed to allow combined sewage to discharge directly to the Missouri River whenever the total flow exceeds the capacity of the interceptor systems. An estimated annual overflow volume of 5 billion gallons (15,300 A-ft) is discharged to the Missouri River.[2/] The annual overflow contains an estimated load of 2000 tons of BOD_5 and 4000 tons of suspended solids. Based upon an evaluation of 20 years of precipitation (1951 through 1970), the volume of overflow exceeds 330 A-ft more than 50 percent of times of overflow occurrence.[4/]

3/ Data obtained from the City of Omaha, Public Works Department.

4/ See Appendix A, Hydrologic Factors.

Chapter V

PLANNING CRITERIA FOR ALLEVIATING COMBINED SEWER OVERFLOWS

Several planning criteria were adopted to provide a basis for comparison of alternative measures for alleviation of the combined sewer overflow problem. These criteria consist of levels of treatment (degree of treatment), design storm recurrence intervals, economic factors, hydrologic assumptions, and the water quality characteristics of combined sewer overflow and storm runoff.

Levels of Treatment

The level of wastewater treatment to be provided is determined by effluent quality requirements. Stricter effluent requirements (lower concentrations of contaminants) can be met by providing higher levels of treatment. For planning purposes, four different effluent qualities were selected in consideration of PL 92-500 and current U.S. EPA guidelines. These effluent qualities and associated levels of treatment, as presented in Table V-1, are not based upon an evaluation of the assimilative capacity of the Missouri River.

Three of the four levels of treatment consist of secondary treatment, tertiary treatment, and treatment to meet background water quality of the receiving stream. These levels are based primarily on PL 92-500 and current U.S. EPA

guidelines. The fourth level, a variance level, was selected because the U.S.-EPA has indicated[1/] that combined sewer overflows may not require a level of treatment as high as secondary. For planning purposes, the variance level is addressed only with respect to treatment processes capable only of producing an effluent of poorer quality than that expected from secondary treatment. Effluent concentration of 50 mg/l BOD_5 and 50 mg/l suspended solids are expected from such facilities.

The cost of providing higher levels of treatment increases significantly from one level to the next. As shown in Plate 8, the total present worth cost of advanced treatment is about twice the cost of secondary treatment; and the total present worth cost of meeting background water quality is triple the cost of secondary treatment.

Secondary Treatment

Secondary treatment of municipal wastewater is capable of limiting effluent concentrations to less than 30 mg/l BOD_5 and 30 mg/l suspended solids. The Nebraska Department of Environmental Control has required the provision of secondary treatment for all Nebraska municipal wastewater treatment plants by December 31, 1975. For cost estimating purposes, secondary level treatment has been assumed achievable by the activated sludge process. Variations of this process and other secondary treatment processes should be considered during later planning stages.

1/ "Secondary Treatment Information", Federal Register, Volume 38, Number 159; August 17, 1973.

Table V-1

EFFLUENT WATER QUALITY
FOR EACH LEVEL OF TREATMENT

Treatment Level	No.	Effluent Characteristics: BOD_5 mg/l	Suspended Solids mg/l	Fecal Coliform per 100 ml	pH	NH_3-N mg/l	NO_3 mg/l	Total Nitrogen mg/l	Total Phosphorus mg/l
Secondary	1	30	30	200	6-9	20	1/	20	10.0
Tertiary	2	15	20	200	6-9	5	15	20	1.0
Background water quality 2/	3	5	2	200	6-9	0.5	4	5	0.1
Variance	-	50	50	1/	6-9	1/	1/	1/	1/

1/ Effluent characteristics not established.

2/ Missouri River quality.

Tertiary Treatment

PL 92-500 states that the best practicable waste treatment technology (BPWTT) shall be achieved by July 1, 1983. For planning purposes the BPWTT is assumed to be tertiary treatment that not only removes more BOD_5 and suspended solids than secondary treatment, but also 90 percent of the ammonia and phosphorus. For cost estimating purposes, this level was assumed to be achieved by provision of two-stage activated sludge for BOD and suspended solids removal and nitrification, addition of alum for phosphorus removal, and sand filtration for additional BOD_5 and suspended solids removal.

Background Water Quality Level

The national goal as stated in PL 92-500 is to eliminate discharge of pollutants by 1985. In anticipation of specific guidelines on meeting this goal, the background water quality level of the Missouri River is assumed to be the required effluent quality. For cost estimating purposes, effluent water quality equal to the Missouri River water quality is assumed achievable with tertiary treatment plus carbon absorption and ion exchange.

Variance Level

The variance level applies to combined sewer overflow treatment facilities which achieve less than secondary level of treatment but greater than that expected by primary settling. For planning purposes, this level is assumed achievable with screening and dissolved air flotation as described in Appendix D.

Hydrologic Factors

The quantity of combined sewer overflows and storm runoff from the study area have been estimated on the basis of selected hydrologic criteria. These criteria, which are presented in Appendix A, are summarized below.

Combined Sewer Overflows

The overflow water from combined sewers during rainfall events with recurrence intervals greater than one-half year is more than 90 percent runoff. For events with longer recurrence intervals, the percentage of overflow that is attributable to runoff is greater than 99 percent. For this reason, the rate and volume of combined sewer overflows in the study area have been estimated on the basis of runoff rates and volumes.

Peak runoff rates and 24-hour runoff volumes were estimated for each area that is tributary to a combined sewer overflow. The results of these estimates, as presented in Table V-2, are based on assumed times of concentration and a determination of the runoff coefficient used in the rational formula. Rainfall intensities for events of 1, 2, 5, 10, and 25 years recurrence intervals were used to estimate runoff and peak flow rates.

The relationship between combined sewer overflows and the flow of the Missouri River is important with respect to determining the impact of combined sewer overflows on the

Table V-2

HYDRAULIC AND HYDROLOGIC DATA FOR OMAHA-MISSOURI RIVER SEWERAGE SYSTEM

Service Area	Area Served, Acres	Junct. Pt. With N-S Interceptor	Outlet Size	Outlet Capacity (cfs)	Interceptor Size Downstream of Junct. Pt.	Estimated Hydraulic Capacity of Interceptor, cfs	Combined Sewage Peak Flows (cfs) 1-yr	2-yr	5-yr	10-yrs	25-yrs	Total 24-Hr Runoff (ac-ft) 1-yr	2-yr	5-yr	10-yrs	25-yrs
Bridge Street	3914		14' x 14'	4000	30" sewer	13	1680	2070	2770	3400	4130	286	359	481	590	711
Mormon Street	316	Morman Street	60" R.C.P.	220	36" sewer	21	275	340	455	560	670	21	26	35	43	51
Minne-Lusa St.	5360	Minne-Lusa St.	12'H x 18'W	5520	72" sewer	87	2930[2]	3590[2]	4760[2]	5660[2]	6700[2]	499	687	826	981[2]	1153
(Relief Sewer)		Cornish Blvd.	108" R.C.P.[1/]	883[2/]	10'-0" x 7'-6" sewer	315	460[2]	560[2]	740[2]	885[2]	1045[2]	78	127	131	153[2]	180
Grace Street	1680	Grace Street	Twin 7'-3" x 9'-0"	1100	66" sewer	66	1235	1510	1990	2365	2780	210	260	346	410	479
	-	Burt Izard St.	Twin 13' x 10'	1390	48" F.M.[3/]	81	-	-	-	-	-	-	-	-	-	-
Burt-Izard Street	2268	Chicago Street	54" brick	130	48" F.M.	81	1895	2310	3050	3615	4245	323	400	530	627	731
	-	Douglas Street	42" R.C.P.	69	48" F.M.	87	-	-	-	-	-	-	-	-	-	-
	-	Farnam Street	3'-6" x 3'-6"	164	48" F.M.	87	-	-	-	-	-	-	-	-	-	-
Leavenworth Street	2061	Jones Street	8'-4"H x 14'W	1340	48" F.M.	91	1605	1960	2590	3070	3610	274	339	450	533	622
		Leavenworth St.	Twin 8'-4" x 10'-0"	1955	66" F.M.	147	-	-	-	-	-	-	-	-	-	-
Pierce Street	100	Pierce Street	42" R.C.P.	92	66" F.M.	150	140	170	230	275	335	11	13	18	21	26
Hickory Street	74	Hickory Street	42" R.C.P.	35	66" F.M.	158	100	120	160	195	240	7	9	13	15	18
Martha Street	212	Martha Street	5'-0" x x 5'-0"	480	66" F.M.	158	215	270	365	445	555	16	20	28	34	42
Grover Street	150	Grover Street	48" R.C.P.	112	66" F.M.	159	135	165	230	290	360	10	13	18	22	27
Riverview Park	618	Riverview Park	8'-0" x 6'-0"	496	66" F.M.	164	430	530	720	865	1050	46	58	80	95	115
Homer Street	65	Homer Street	36" R.C.P.	44	66" F.M.	164	85	100	140	165	200	6	8	11	13	15
Missouri Ave.	386	Missouri Ave.	5'-0" x 5'-0"	297	66" F.M.	166	255	315	430	530	640	28	35	48	58	70
South Omaha	2116	-	Twin 10'-6" x 12'	3278	-	-	1315	1610	2135	2540	2995	224	283	371	441	516
Monroe Street	1760	-	Twin 96" R.C.P.	2902	-	-	1215	1480	1963	2330	2745	207	256	341	404	473
"U" Street	33	-	48" R.C.P.	170	-	-	45	60	75	90	110	4	4	6	7	8
TOTALS	21,113			20,677			12,135	17,160	22,803	23,880	28,280	1964	2897	3733	3857	4526

Notes:
1 utlet into the North Interceptor instead of into the Missouri River
2 mated flow rate
3 F.M. = Force Main

Missouri River. During the navigation season, the flow of the Missouri River is maintained at about 30,000 cubic feet per second (cfs). During the winter months when navigation ceases, the flow is reduced to about 10,000 cfs. Because the Missouri River flow is regulated, it is possible to make a general statement about the relationship. More than 90 percent of the overflow events in Omaha occur during the navigation season. In addition, all overflow events with recurrence intervals equal to or greater than one year also occur during the navigation season when the flow of the Missouri River is 30,000 cfs.

Storm Water Runoff

Several areas in the study area have separate storm sewers. The volume of runoff from each of these areas is based on the same type of analysis as was done for the combined sewer overflows. A runoff coefficient was determined and the volumes for an event with a 25-year recurrence interval. Peak runoff flow rates were not estimated. The volumes of runoff from each separate storm sewered area are presented in Table V-3.

Water Quality Criteria

Characterization of the water quality of combined sewer overflows provides a basis for formulating and evaluating various pollution abatement alternatives. The results of a literature search to characterize both combined sewer overflows and urban storm runoff are summarized in Appendix A and

presented in Appendix B. Even though combined sewer overflow is mainly urban storm water runoff, the water quality of combined sewer overflow can greatly differ from the water quality of separate storm water runoff.

Table V-3

SOURCES OF URBAN RUNOFF FROM STORM SEWERS

Location	Receiving Body of Water	Drainage Area, Acres	Estimated 24 Hour Runoff Volume, A-ft		
			(1yr)1/	(10yr)1/	(25yr)1/
Bridge Street Service Area	Missouri River	3914	286	590	711
Eppley Airfield	Carter Lake	600	40	80	95
Eppley Airfield	Missouri River	470	7	19	24
Northwest of Carter Lake	Carter Lake	450	32	66	80
Leavenworth Service Area	Missouri River	95	12	25	39

1/ Recurrence interval.

Combined Sewer Overflow Characteristics

Pollutant concentrations in the overflow from combined sewers are relatively high at the beginning of a runoff event, but generally begin to decrease within the first two hours of

runoff. Thereafter, the concentrations decrease rapidly as the sanitary sewage becomes more diluted. If the runoff continues for more than 12 hours, the concentrations usually level out for the remainder of the event. The initial high concentration of pollutants is termed the "first flush" and is reported by most researchers.

The water quality characteristics of combined sewer overflows are reported to vary from runoff event to runoff event, overflow point to overflow point, and city to city. However, for planning purposes, concentrations of 100 mg/l BOD_5 and 250 mg/l suspended solids are assumed to represent average concentrations during any runoff event. Initial concentrations of BOD_5 and suspended solids are assumed to be 250 mg/l and 400 mg/l, respectively, during the first two hours of runoff. As shown in Plate 12, these concentrations are assumed to taper off to 50 mg/l BOD_5 and 200 mg/l suspended solids.

The amount of pollutants contained in a combined sewer overflow depends upon several factors. These factors are the quantity of surface material washed into the sewers, the quantity of settled matter that is scoured and picked up by the storm flow, and the percentage and strength of the dry-weather flow component in the overflow. Each of the factors may be dependent upon the rainfall intensity and the time between runoff events.

The rate at which rainfall washes loose particulate matter from street surfaces depends upon the rainfall intensity, street surface characteristics, and particle size.

The wash-off phenomenon has been simulated by a simple, exponential equation[2/]:

$$Nc = No\ (1 - e^{-krt})$$

where Nc is the weight of material of a given particle size washed off a street having a loading of No after t minutes of rainfall at an intensity of r inches per hour. The proportionality constant k depends upon street surface characteristics. Because the equation is exponential, rainfall intensities greater than a certain value will not substantially affect the amount of material washed off the street. The build up of material, No, will depend upon the time between street washing (rainfall or street sweeping).

The amount of material scoured within the combined sewer is dependent upon the velocity of the combined sewer flow and the amount of material that has been deposited since the last runoff event. At this time, a quantitative relationship between rainfall intensity and frequency and the amount scoured has not been established. However, the higher rainfall intensities will result in higher flow velocities that, in turn, will resuspend more settled material. Once the sewers have been scoured, the effect of higher intensity rainfalls will be negligible. Time between runoff events will certainly affect the amount of material to be scoured. Early spring runoff events can be expected to contain the heaviest pollutant loads of all the estimated 50 annual overflow events.

2/ Sartor, J.D., and Boyd, G.B., Water Pollution Aspects of Street Surface Contaminants, EPA-R2-72-081, November, 1972.

All combined sewer overflows will contain some dry-weather sewage component. The maximum percentage that can be contained in the overflow is dependent upon the diversion structure. Along the interceptor system in Omaha, the diversion structures prevent discharge of overflow until the combined sewage contains less than a set percentage of sewage. Consequently, during a low intensity runoff event, the overflow can contain 20 percent sewage along the North Omaha Interceptor or 33 percent sewage along the South Omaha Interceptor. As the rainfall intensities increase, the percentage of dry-weather sewage component decreases. For events with recurrence intervals greater than one year, however, overflows contain less than one percent dry-weather sewage. Although almost all of the dry-weather sewage component is discharged in the overflow, it is diluted almost 100 times during the one year recurrence interval event. Finally, the percentage of dry-weather component in the combined sewer overflow is, therefore, independent of the time between runoff events.

Urban Storm Water Runoff Characteristics

Urban storm water runoff carries a lower concentration of BOD_5 than does combined sewer overflow, but it reportedly carries a higher suspended solids concentration. Average BOD_5 and suspended solids concentrations are assumed to be 18 mg/l and 400 mg/l, respectively. For purposes of this study, it was assumed that urban storm water runoff could be discharged to a receiving water without treatment.

One drainage area, which has separate storm sewers but is not urban, is the airport, Eppley Airfield. Characteristics of the water quality of its runoff may be substantially different than those assumed for this study. Runoff from the runways could contain deicing agents and hexane solubles such as oil and grease. These substances may cause pollution of the receiving water, and consequently, may have to be removed prior to discharge. Control measures should be applied if this storm water discharge proves to adversely affect water quality.

Selection of Recurrence Interval

The magnitude of a precipitation event and its resulting combined sewer overflow is indicated by the recurrence interval between equal or greater events. For longer recurrence intervals, the corresponding events are larger.

Overflow of combined sewers is caused by rainfall and snowmelt and the resulting runoff from areas tributary to the combined sewers. Prevention of combined sewer overflows caused by the 1-year rainfall event will also prevent combined sewer overflow caused by any snowmelt event because of the relatively high intensity of the rainfall event.

In the Omaha area, there are about 50 combined sewer overflows each year. By constructing a device to handle and treat runoff from a storm with a recurrence interval of one year, about 98 percent of the overflows (49 out of the 50)

would be prevented. However, all storms of larger magnitude, such as the 2-year recurrence interval storm, would cause an overflow of mixed sanitary sewage and storm water to the river. Selection of a recurrence interval for evaluation and comparison of alternatives involved consideration of how often an overflow of raw sewage is tolerable and whether control to that recurrence interval is economically justifiable.

Determination of a recurrence interval on an economic basis is dependent upon the assignment of monetary value to the benefits derived. It is not possible, within the context of this study, to assign a monetary benefit to the reduction in the number of combined sewer overflows. Intangible factors, however, such as aesthetic qualities and public willingness to pay were taken into account in the process of selection.

The incremental cost of preventing one additional overflow event is a concept of cost-effectiveness that has been used for selection of a design recurrence interval. If a selected design recurrence interval would allow 50 overflows in 100 years (2-year recurrence interval) and the additional cost of reducing the overflows to 49 in 100 years was $1,000,000 more, then the incremental cost of preventing one more overflow for the 2-year recurrence interval is $1,000,000. If this incremental cost were $10,000,000 for the 50-year recurrence interval, it would appear that preventing one additional event for the 50-year recurrence interval level would be disproportionately high.

A graphical procedure was used to compare and evaluate different recurrence intervals. The number of overflows allowed by designing for a particular recurrence interval is presented in Plate 9. This theoretical relationship is shown for a 100 year period in which there would be about 5000 overflows (50 per year times 100 years). By designing for the 1-year recurrence interval, the number of overflows would be limited to about one per year or 100 in the 100 year period. The provision of a design for a 2-year recurrence interval allows 50 overflows in 100 years. Thus, the 2-year design allows 50 overflows less than the 1-year design, but 30 more than the 20 allowed by the 5-year design.

As shown in Plate 10, the cost of providing capacity to handle larger recurrence intervals increases for all alternatives considered. The incremental cost of designing for longer recurrence intervals and preventing additional overflows can be extracted from the curves on Plate 10. By dividing the incremental cost by the number of additional overflows prevented, an incremental unit cost of additional protection is established. As shown in Plate 11, the resultant curves of incremental unit cost versus total present worth cost indicate that for recurrence intervals above five years, the incremental cost of preventing additional overflows rises rapidly and there is diminishing return for incremental investment.

Although the optimum design recurrence interval was not determined, a recurrence interval of five years was selected

for comparison of alternative measures to alleviate the combined sewer overflow problem. The selection of a design recurrence interval is not only dependent upon the cost of the project, but also on the necessity of meeting established standards and on the impact on the environment. Design recurrence intervals greater than five years appear to have an excessive incremental cost for additional protection and to have little or no justification in view of the high assimilative capacity of the Missouri River. The selection of a 5-year recurrence level interval is also consistent with the EPA's draft guidelines for Areawide Planning under Sec. 208 of PL 92-500 that indicate that recurrence intervals of from 1- to 5-years should be evaluated for intermittent point sources of pollution.

Cost Evaluation

Alternative plans for the abatement of pollution from combined sewer overflows are evaluated and compared primarily on the basis of cost. Federal EPA guidelines for wastewater management planning were followed in selection of the cost criteria. A discussion of the selected cost criteria is presented in Appendix A and summarized below.

Cost Criteria

The following criteria were used to evaluate the alternatives and to compare them on the basis of total present worth

cost. The criteria consist of a planning period from the present to 1995; an interest rate of seven percent; an economic life of 50 years for sewers, force mains, and storage basins, and an economic life of 25 years for pumping stations, lift stations and sewage treatment facilities.

Costs

Cost estimates were prepared for each of the alternatives. The construction costs for each alternative include the initial installation cost plus 35 percent for engineering, legal and administrative fees, and contingency. Costs for system components are based on an ENR Construction Index of 2000, representative of costs as of March, 1974.

Land acquisition costs are included in the total present worth cost for each alternative. The unit costs for land in various areas are presented in Appendix A.

Chapter VI

PRELIMINARY STUDY OF TWELVE FEASIBLE ALTERNATIVES

Overflow from combined sewers is now recognized as a major source of pollution. Only during the last twenty years has this intermittent source of pollution been investigated to characterize its waste load and to evaluate ways to reduce its impact on the receiving stream. Because abatement of pollution from combined sewer overflow had relatively little impetus prior to the passage of PL92-500, the number of combined sewer overflow treatment facilities in actual operation is quite small. Thus, the solution to the combined sewer overflow problem may involve construction of a system unprecedented in the Omaha area.

A number of planning steps precede the final design of a combined sewer pollution abatement system. These steps are necessary to ensure that the constructed facility will meet all of the desired objectives. These objectives include cost-effectiveness, ability to reliably meet the selected level of performance, compatibility with the environment, and minimal adverse social, economic, and environmental impact. The planning steps leading to final design is the planning strategy.

The planning strategy adopted for this study involves formulation and selection of alternative concepts, preliminary evaluation of alternative concepts, selection of alternative concepts to be evaluated as alternative plans, and evaluation of alternative plans. This strategy allows

consideration of a great number of concepts and proportions the degree of evaluation to a concept's potential for being selected for the final design.

Basic Components of Alternatives

Combined sewer overflows are characteristically of short duration with high rate discharges of polluted water. Pollutional load of combined sewer overflows can be reduced either by reduction of the pollutants that enter the combined sewer, elimination of the combined sewer, or treatment of the overflow.

While urban housekeeping is an important factor, Sartor and Boyd[1/] have reported that street sweeping can only reduce the amount of pollutants entering the combined sewers by less than 50 percent. Elimination of combined sewers is not a basic component of an alternative, but a concept in itself.

Treatment of the combined sewer overflow raises several practical problems because of the nature of the overflow. Treatment capacity must either be sized to handle the peak flow during the short overflow period or the overflow rate must be attenuated by storage to allow treatment at a lower rate. Consequently, storage and conveyance become the other two components of alternatives to abate pollution from combined sewers. Of the three components, treatment has received the greatest attention from researchers.

1/ Sartor, J.D. and Boyd, G.B., Water Pollution Aspects of Street Surface Contaminants, EPA-R2-72-081, November, 1972.

Treatment of Combined Sewer Overflow

Treatment components can be divided into two classes: those that handle the unattenuated peak flow and those that are coupled with some storage component. Treatment devices that receive the unattenuated peak flow characteristically provide less than secondary level treatment. The effluent quality from these high rate treatment processes (flow-through treatment) falls in the range between secondary and primary level treatment. On the other hand, treatment processes that receive regulated flow from a storage facility can provide any desired effluent quality.

Flow-through treatment. Several treatment processes designed to handle unattenuated peak flows have been developed. These processes rely on physical-chemical processes to remove pollutants. Also included in this group of treatment processes are the methods of disinfecting combined sewer overfows.

Screening and dissolved-air flotation units have been tested in Milwaukee, Wisconsin, and elsewhere. The unit screens coarse solids and, by injecting air into the water, causes much of the colloidal and suspended matter to float and be skimmed off. As indicated in Appendix D, the effluent from these units would generally not meet the requirements for secondary treatment.

High-rate filtration has been demonstrated at Cleveland, Ohio, to provide less than secondary level treatment when used with a polyelectrolyte. Flow passes through a coarse screen and then through a deep bed of anthracite coal and sand. This treatment facility can also be used to polish an effluent from a secondary level treatment plant. In that case, the coarse screen would be unnecessary.

Microstraining is a tertiary waste treatment process conventionally used to polish secondary effluent. Flow enters the open end of a partially submerged, rotating drum screen. It then passes through the fine mesh screens that retain the solids. As the drum rotates, the part of the drum above the water surface is sprayed with water to remove solids from the screen. This process has been successful in reducing suspended solids concentrations of secondary effluents to 10 mg/l, but it has not been demonstrated to be effective for treatment of combined sewer overflow.

The swirl flow regulator/solids-liquid separator provides primary level treatment of combined sewer overflow. This simple device is designed to separate settleable and light weight organic suspended matter from the overflow at a fraction of the detention time required for conventional sedimentation. Flow is directed into the annular shaped device to create a swirl action. The low-flow concentrate is diverted to the sanitary sewer system and the overflow is discharged.

High-rate disinfection has been demonstrated in Philadelphia, Pennsylvania, to provide 99.99 percent coliform kill with chlorine dosages of 10 mg/l and a detention time of two minutes. The high kills are achievable with short detention times and low dosages because of the intimate contact between overflow and chlorine provided in contact tank specially designed to produce a very high degree of turbulence.

Treatment of attenuated flow. Treatment processes in this category include variations of the activated sludge process, aerated lagoons, trickling filters, physical-chemical processes, and specific unit processes that are used with secondary level treatment plants to obtain higher levels of treatment (tertiary and/or advanced.)

The activated sludge process is a biological treatment process that employs micro-organisms to remove the organic wastes. These organisms are mixed with the wastewater and oxygen is added. After several hours of contact, the organisms are settled out of the water and returned to be mixed with new, incoming wastewater. After clarification and disinfection, the wastewater is discharged as a secondary level effluent. The process has several operating mode variations, depending upon the characteristics of the raw waste. Dilute wastes, such as partly stabilized combined sewer overflow from an aerated storage basin, can be treated in an activated sludge process without upset; but some modification to the normal operating mode may be required.

The activated sludge process can be readily upgraded to provide higher levels of treatment. Nitrogen removal can be accomplished with two-stage nitrification and a denitrifying filter. Further BOD_5 and suspended solids can be removed with carbon absorption and sand filtration.

The trickling filter is another biological treatment process similar to the activated sludge process except that the micro-organisms are attached to a fixed medium. The wastewater is distributed over a stone or plastic medium upon which the micro-organisms are attached. In a variation of this process the medium is a plastic disc that rotates in a trough of flowing wastewater. The trickling filter and its variations are slightly more adaptable to variations in the inflow rate than the activated sludge process.

Aerated lagoons, mechanically aerated with floating aerators, can provide the equivalent of secondary treatment. This biological treatment process requires long detention times of at least ten days to achieve 85 percent BOD_5 removal. This treatment process can be easily coupled with storage of the combined sewer overflow. Unlike the activated sludge process and the trickling filter, the aerated lagoon efficiency is less dependent on the influent constituent levels.

In Albany, New York, raw sewage was mixed with powdered activated carbon, coagulated with alum, settled with polyelectrolyte, and filtered through a multi-media filter. Removal

efficiencies between secondary and tertiary level treatment were achieved. This process also appears to be adaptable for treatment of combined sewer overflows.

Disposal of secondary level effluent on land by spray irrigation is an alternative to discharging to a water course. Spray irrigation can be economically competitive compared with providing advanced waste treatment to meet background water quality. Land disposal beneficially utilizes nutrients that otherwise would be detrimental to surface water bodies and is also an attractive method in areas where there is a shortage of water for agriculture.

Storage of Combined Sewer Overflow

Storage of the combined sewer overflow is necessary if the overflow is to receive a minimum of secondary treatment. Storage can be constructed in excavated or diked earthen reservoirs, steel or concrete tanks, rubber storage bags, or mined underground caverns. In addition, combined sewage can be stored in the sewer pipes, or storm runoff can be stored before it enters the combined sewer system. In the study area, the land use and topography influences the attractiveness of individual storage components.

Earthen Reservoirs. Earthen reservoirs, at or near the ground surface, could provide relatively inexpensive storage of combined sewer overflow. Several construction possibilities exist for an earthen reservoir in a particular location.

Large diked reservoirs with earth embankment sides, rising to 15 to 20 ft above the ground surface, could

be constructed in large open space areas. They would provide the least expensive storage volume because the only excavation required would be to provide fill material for the sides. Such reservoirs would have low profiles because of their large surface area. The earth embankments would be maintained with grass cover and could be landscaped for esthetic quality.

Diked reservoirs constructed as described above would cost approximately $4000 per A-ft of volume for a representative total volume of 1000 A-ft. This cost is based on construction of a circular reservoir with 15 ft embankments. Reservoirs of larger volume will cost less per A-ft of storage volume.

Diked reservoirs constructed along the Missouri River could utilize the existing levee for the west side. Some excavation would be required in addition to construction of embankments. Part of the excavated material could be used for the construction of the embankments, but some would have to be disposed of. The area between the levee and the Missouri River is undeveloped floodway. Construction of a storage reservoir in the floodway would constrict the floodway and might necessitate a slight raising of upstream levees to maintain the present level of protection. The exact location of these reservoirs would depend upon the desired volume and potential conflict with the Riverfront Development Program.

The cost of this type of storage is approximately 2.5 times the cost of the ground level reservoirs previously described

because of high ground water. The estimated cost for a volume of 2000 A-ft is about $6750 per A-ft.

The most expensive type of earthen reservoir is excavated below the existing grade. This type of basin could be located just about anywhere in the study area. If located near the Missouri River, there would be an added cost to overcome the high ground-water table. Excavated basins, 15 feet deep with 2.5:1 side slopes, are estimated to cost about 10 times the cost of the ground level diked reservoir. With a clay lining and under drains, a 100 A-ft excavated storage basin is estimated to cost $17,500 per A-ft.

Steel or concrete tanks. In areas where open storage, such as earthen basins, is undesirable or where it is desirable to locate storage below ground, concrete or steel tanks can be used. These structures can be buried where needed and remain out of sight. Unfortunately, when these tanks are empty, they may be subject to upward buoyancy forces caused by the high ground water. Buried storage facilities, where there are no problems with buoyancy forces, are estimated to cost about $27,000 per A-ft of storage volume (for a 100 A-ft facility). The same buried storage volume constructed along the Missouri River would cost about $190,000 per A-ft.

Mined storage. Mined storage is an alternative to buried storage. Located several hundred feet below the

surface, the storage volume would be mined out in sound rock formation. This type of storage is relatively expensive, but well below the cost of buried storage where high ground water conditions exist. For storage between 2000 and 5000 A-ft, the unit cost is about $27,000 per A-ft.

Collapsible bag storage. Collapsible rubber bags located in the Missouri River could be used to retain the combined sewer overflows. Large, flexible containers would be anchored to the river bottom in collapsed form until overflows were discharged to them. After an overflow event, these containers would be pumped out. Cost for this type of storage is estimated to be $120,000 per A-ft.

In-system storage. The storage of surface runoff before it becomes combined sewer overflow can be accomplished by utilizing the volume of the sewer pipes for storage. Controlled release of the overflows to the river will cause the combined sewage to fill the sewer pipes during periods of runoff. Computer regulated controls to operate overflow devices near the combined sewer outlet have been installed in several cities and their performance is being studied.

Storage of rainfall before it enters the sewer system can also be accomplished by utilizing rooftop storage, parking lot storage, and porous pavements. Storage by these methods is particularly attractive in areas undergoing redevelopment or newly developing areas.

Conveyance of Combined Sewer Overflows

Combined sewer overflow outlets are rarely located in places suitable for inexpensive storage or places with sufficient area for a treatment facility. Thus, conveyance facilities are often a component of an alternative concept or plan to abate pollution from combined sewers. Conveyance facilities include gravity sewers, force mains, open channels, and tunnels.

Gravity sewers. The flow carrying capacity of a gravity sewer depends upon its diameter, its slope, and the coefficient of roughness associated with its material of construction. Of these three variables the coefficient of roughness will have the least effect on the capacity of a gravity sewer to convey combined sewer overflow. Steeper slopes allow higher carrying capacities, but mean increased depths of excavation, higher costs, and additional lift stations for conveyance over a long distance. The slope can be decreased and capacity maintained if the diameter is increased, but this also increases the cost significantly.

Gravity sewers are often the least expensive method of conveying wastewater. However, when the necessary conveyance capacity requires sewer diameters greater than 6 ft, other methods of conveyance may be less expensive. Use of gravity sewers to convey peak flows from the combined sewer overflows to a storage facility is very expensive, because of the large diameter and steep slopes required.

Force Mains. Conveyance of wastewater under pumping pressure through a force main is normally used when gravity sewers require excessive depths to maintain the necessary slope or when the discharge point is at higher elevation than the starting point. Force mains are not, however, practical for conveyance of peak combined sewer overflows to a storage facility. The required pumping power that must be available upon demand for the short, intense peak flows is extremely high. Force mains are practical, however, for conveyance of stored overflow to a treatment facility.

Open Channels. Open channels are an alternative to gravity sewers. Where there is adequate slope and available land, open channels may be used to convey high peak flows at a lower cost than a gravity sewer. Open channels are rarely suitable or practical for conveyance in wastewater flows that require gravity sewer diameters less than 6 ft.

Tunnels. Tunnels for conveyance of wastewater have been constructed in several European and U.S. cities. These tunnels may be constructed near the surface or up to several hundred feet underground in sound rock formations. The structural integrity of some rock formations can allow construction of tunnels up to 40 ft in diameter. This size is prohibitively expensive for a conventional gravity sewer near the ground surface. Recent construction methods have reduced the cost of tunneling so that tunnels with diameters greater than 6 ft may be competitive with gravity sewers.

Formulation of Conceptual Components

Alternative concepts can be formulated by combining the basic components. The number of possible alternative concepts becomes far too great to evaluate unless many are eliminated from consideration on the basis of a brief review. Several alternative concepts may then be attractive for evaluation. Based upon that evaluation, a select few may then be considered as alternative plans. The initial process of screening the numerous possibilities and the results of the screening are discussed in the following sections.

Initial Screening

More than thirty alternative concepts were fomulated on the basis of combining different components. Under this section, only those concepts eliminated during the initial screening are discussed. Some alternatives are eliminated because of technical infeasibility. Others are eliminated because of cost considerations, and still others because they could not perform to the desired minimum level. The concepts as listed in Table VI-1 are still a partial list, but they do represent a fair cross-section of alternative concepts eliminated by initial screening. The alternative concepts listed in Table VI-1 are described in terms of their basic components. Concepts with flow-through treatment do not have storage or conveyance components.

Table VI-1

INITIAL ALTERNATIVE CONCEPTS ELIMINATED FROM FURTHER CONSIDERATION

Concept Description	Reason for Elimination from Further Consideration
High-Rate Filtration at overflow points	Technically and economically unattractive.
Micro-straining at the overflow points	Technically and economically unattractive.
Swirlflow separator	Technically inadequate to meet minimum desired level of treatment.
High-rate disinfection	Inadequate treatment if used alone.
Biological or physical-chemical treatment as a part of any scheme with storage	All treatment facilities can be used comparably for most alternatives so that the selection of a particular treatment process will not favor one alternative concept over another.
Storage in collapsible bags	Number of bags required is excessive - too costly.
Storage on rooftops, parking lots, etc.	Inadequate storage available - too costly to provide now.
Conveyance by gravity sewers to large storage reservoirs	Gravity conveyance unpractical because of large diameters and excessive slopes required.
Conveyance by forcemain to large storage reservoirs	Pumping stations and power requirements too costly for pumping peak overflow rates to storage.

Table VI-1 (Continued)

INITIAL ALTERNATIVE CONCEPTS ELIMINATED FROM FURTHER CONSIDERATION

Concept Description	Reason for Elimination from Further Consideration
Storage of overflow in quarry reservoir	Quarry is located too far away to be practical.
Pump stored overflow to treatment facility in new force main, parallel to existing interceptor system	Ignores unused capacity of existing interceptor system.
Pumped storage facility as part of mined storage scheme	Not enough electrical demand close enough to study area to be practical.
Concrete storage tanks at airport, under runways	Distance too far to convey flows by gravity and storage costs would be excessive.
Storage in deep tunnel conveyance by enlarging tunnel diameters	Inadequate storage in tunnel system and cost of tunnels as a storage facility is too expensive.
In-stream aeration	Will not meet PL92-500 regulations.

Alternative concepts that differ only in selection of a treatment process were eliminated from further consideration because the selection of a particular treatment process would have an insignificant impact in the comparison of these alternatives. Although some of these alternative concepts are eliminated, the refinement of selected alternative plans may still incorporate components of these alternative concepts. An example might be the use of a swirl flow separator to decrease the size and/or cost of a treatment component in a selected alternative plan.

Alternative Concepts

Twelve alternative concepts (not shown in Table VI-1) were retained after the initial screening process. These concepts represent a range of solutions that appear to be technically feasible, environmentally compatible, and economically reasonable.

Several general assumptions regarding the sizing of the components were made to facilitate the comparison of the alternative concepts. These assumptions concern storage and pump-out facilities, treatment facilities, and the basis for sizing of components. Each alternative, except for sewer separation, is sized and costed to provide the equivalent of secondary treatment for overflow events with recurrence intervals of five years or less. Total present worth costs are based upon a 50-year period and interest rate of 7 percent.

The various types of storage facilities studied included: open excavated surface reservoirs, open ground level diked reservoirs, covered and buried concrete tanks, and mined underground reservoirs. In most cases the storage facilities would be equipped with surface aerators to maintain aerobic conditions, prevent odors, and provide sufficient mixing of solids for eventual discharge to treatment facilities.

Storage is sized to provide enough volume to contain the first 48 hours of runoff. It is assumed that evacuation of the storage facility will be initiated within the first 24 hours of the event and will be at a high enough rate to ensure storage of subsequent runoff events during the evacuation period.

The maximum rate at which the overflows would be evacuated from the storage facilities is assumed to be the maximum rate at which the existing interceptor could accept the stored wastewater during periods of dry weather flow. This maximum rate is estimated to be 90 cfs, or 60 mgd. It is further assumed that reservoir evacuation would be confined to a period of 18 hours of each 24 hour day in order to permit use of less costly off-peak electric power. Flows also would be pumped during dry weather, low flow periods to match the peak hydraulic design capacity of the Missouri River sewage treatment. Based on the above rate, the maximum evacuation time for a 5-year recurrence interval is approximately 20 days.

The future expansion of the existing Missouri River primary treatment plant will provide for secondary treatment of the

dry-weather sanitary sewage. For most of the alternatives studied, it was assumed that some of the stored combined sewer overflows could be handled by the treatment plant during dry-weather low-flow periods. It was also assumed that the remaining stored overflows would be handled by additional capacity or further expansion at the proposed activated sludge treatment facilities beyond the proposed 65 mgd capacity. An additional 20 to 30 mgd treatment capacity may have to be provided for handling the remaining stored overflow.

Advanced waste treatment processes, using multi-media filtration, carbon absorption, and ion-exchange could be added to the treatment plant facilities to achieve a higher degree of treatment. However, the cost of these facilities would be common to all alternative concepts providing secondary treatment. Therefore, the cost of providing higher levels of treatment are not required to compare alternatives.

Areas that were not considered as contributing to the total combined sewer overflows are the Carter Lake area and Eppley Airfield. Separate sewers are recommended throughout these areas.

The performance of each alternative concept during events of larger magnitude than the design basis is approximately the same. An estimate of the BOD_5 concentration in the overflows caused by events that exceed the design recurrence interval is shown on Plate 13, based on capturing the "first flush" effect in the storage facility. A conservative estimate of

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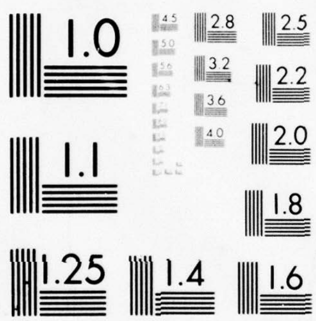
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1.4
1.6

the overflow BOD_5 concentration would average between 50 and 100 mg/l depending on the recurrence interval of the overload event. The actual concentration is likely to be lower since some retention would take place in the aerated storage facility prior to discharge of the overflow.

The twelve alternative concepts considered and discussed in the following sections are listed below:

1. Buried Storage at Outfalls.
2. Diked Storage Along Levee.
3. Upstream Retention.

4a. Deep Tunnel North to Ground Level Storage.

4b. Excavated Storage North - Deep Tunnel South to Ground Storage.

5a. Deep Tunnel With Mined Storage.

5b. Excavated Storage North - Deep Tunnel to Mined Storage South.

6. Flow-Through Treatment With Storage at Outfall.
7. Sewer Separation.

A. In-system Attenuation Devices.

B. Deep Tunnel to the Papillon Creek S.T.P.

C. Flow-Through Treatment for "First-Flush".

Alternative concepts are evaluated and judged on the basis of total costs and intangible, subjective considerations.

The costs of the alternatives studied are summarized on Plate 14. Subjective evaluations, such as likelihood of public acceptance, disruptive effects, etc., are summarized for all the alternatives on Plate 15.

Alternative 1 - Buried Storage at Outfalls - This alternative uses buried concrete storage tanks located at the major overflow points along the main interceptor route. The stored overflows would be discharged to the existing interceptor system and treated at the upgraded Missouri River treatment plant. This alternative is illustrated on Plate 16, which shows approximate locations of the buried storage tanks.

The storage tanks would be sized according to runoff volume from each of the service areas. Industrial tracts and large parking lots could serve as potential sites for the buried storage tanks. The overflows would be diverted to the buried storage facilities, aerated, and stored for a maximum of 20 days. The tanks would be evacuated by pumping to the existing interceptor system at a maximum rate of 60 mgd. The overflows would then receive necessary treatment at the expanded Missouri River treatment plant. The buried storage facilities might use diffused air instead of surface aerators due to possible lack of headroom and restrictions in available freeboard. During dry weather periods, the storage tanks could be emptied and cleaned. Screening equipment would be provided at the tank inlet for removal of large solids. A typical storage tank would be 600 ft square ranging from 25 to 30 ft deep with approximately three to

five feet of cover below the existing grade. Storage requirements for the 5-year recurrence interval would average 200 acre-feet per basin.

The construction of buried tanks at the individual outfalls would have a short-term local environmental impact due to increased traffic of construction vehicles. The major impact of the construction of holding tanks would be the disposal of several million cubic yards of spoil material, consisting mainly of coarse and fine grained sand. If construction uses could not be found for the material (e.g., levees, roads, landfill), it would have to be placed at a disposal site selected for minimum environmental impact.

This alternative would entail estimated initial construction costs of $536 million and total annual operation and maintenance costs of $3.3 million. The estimated total present worth cost is $589 million. This cost estimate includes construction, operation, maintenance, additional treatment facilities at the Missouri River plant, and land acquisition. The individual component costs are shown on Plate 14. The storage costs for this alternative are high because high ground water conditions would necessitate expensive construction features to overcome buoyancy forces when the tanks are empty. Land acquisition costs would be low since storage facilities could be located under existing parking lots and industrial tracts.

This alternative has the advantage of eliminating the need for additional grit removal and by-pass facilities scheduled for construction at the existing outfall points. If

the grit removal facilities are shut down for maintenance, dry weather by-passes and wet-weather overflows could be discharged to the buried storage facilities instead of to the Missouri River. In addition, the basins could be used for temporary storage of dry-weather flows if a failure or shut down should occur at the Missouri River Sewage Treatment Plant. Other advantages include negligible visual impact on the area since the facilities are buried. Disadvantages of this alternative are the extremely high costs and the problem of obtaining necessary sites for the buried facilities.

Alternative 2 - Diked Storage Along Levee - This alternative would utilize diked, open storage reservoirs located in the Missouri River floodway outside the flood control levees. These elongated reservoir areas, located on the river side of the levee, would store the overflows for release to the existing interceptor system and treatment at the upgraded Missouri River treatment Plant.

It is estimated that at least three reservoirs would be required to retain the combined sewer overflows from the study area. The two northern reservoirs would serve the Minne-Lusa and Grace Street areas. The third reservoir would serve the southern portion of the study area. The approximate locations of the reservoirs are shown on Plate 17.

The reservoirs would resemble channels approximately 200 to 300 ft in width and one to three miles in length with 15 to 20 ft of liquid depth. Trashracks and screens would be utilized at the outfalls to remove large debris. Overflows would

be lifted to the reservoirs from newly constructed outfall overflow sumps by low lift screw pumps. Some of the outfalls would be interconnected by open channels or relief sewers to minimize the number of screw pump facilities required. Outfall points presently utilizing diversion channels such as the Grace Street channel, would be expanded. A minimum amount of permanent liquid depth would be maintained to provide a liquid seal to prevent odor-producing conditions.

The reservoir channels would be equipped with floating mechanical aerators to maintain aerobic conditions and suspension of lighter organic solids. During dry-weather periods, a minimum number of aerators would be used to reduce power usage. Some permanent deposition of heavier inorganic grit material would occur. Periodic dredging (once in 10 years) may be required to restore the reservoir capacity. The stored overflows would be pumped back into the existing interceptor system for treatment at the Missouri River plant, as described in Alternative 1.

This alternative would encroach on the Missouri River floodway causing a rise in flood water levels upstream. It is estimated however, that an increase in the levee heights throughout the Omaha area to compensate for the decrease in width in the floodway would be negligible.

Creation and maintenance of diked storage in the narrow strip between the existing levee and the river would preempt the use of the land for other purposes such as a roadway or

park. The land currently is undeveloped, but heavily disturbed ecologically, due to the construction of levees and outfalls, and to flooding and erosion. The storage reservoirs would not be highly visible from ground level but nevertheless would probably have unfavorable aesthetic impact.

This alternative would entail an estimated initial construction cost of $126 million and total annual operation and maintenance costs of $4.1 million. The estimated total present worth cost is $203 million. These costs include construction, operation and maintenance, lift stations and screw pump installation, additional treatment facilities at the Missouri River plant, levee adjustments, and land acquisition costs. A summary of all costs for the 5-year recurrence interval is shown on Plate 14.

This alternative would have the advantages of eliminating the need for duplication of grit removal facilities at the outfall points and providing for storage of dry-weather flow in case of treatment plant failure. It is attractive economically for both capital and operation and maintenance costs. Disadvantages are unfavorable visual impact and the effect on the floodway, requiring modifications to the existing levee system throughout the area.

<u>Alternative 3 - Upstream Retention</u> - This alternative would utilize a multitude of upstream retention sites for storage of overflows followed by release of the stored overflows to the

existing main sewer and interceptor system and treatment at the upgraded Missouri River plant. It was not practical within the scope of this study to specifically identify potential sites, but a number of locations are shown schematically on Plate 18.

Overflows would be diverted from the existing sewers through diversion structures, channels, or additional relief sewers into the storage retention basins. The storage basins, wherever possible, would be open excavated reservoirs and the remaining reservoirs would be buried concrete tanks with three to five feet of cover. Existing parking lots, city parks, lagoons, or expressway intersections could be used as storage sites. An estimated 25 storage sites would be required. The retention basins would average approximately 500 ft square and 20 ft deep, utilizing 6.5 surface acres. Each of the storage basins would be equipped with a pumping station to release the stored overflows back to the existing conveyance system. During extended dry-weather flow periods the reservoirs could be emptied and cleaned.

An environmental impact of this alternative would be the disruption of the areas in which the underground storage reservoirs would be built. Disruption during construction and the consequent loss of trees and shrubs would be major impacts with broad secondary effects of life-style, property values, and land use. Open excavated reservoirs would have unfavorable aesthetic impact.

This alternative would entail an estimated initial construction cost of $137 million and total annual operation and maintenance costs of $3.6 million. The estimated total present worth cost is $206 million. These costs include construction, operation, maintenance, pumping stations, treatment, and land costs. This alternative has the highest land costs, estimated at $12.2 million.

This alternative's only real advantage is that the cost is competitive with the other alternatives. However, it has the disadvantage of poor aesthetic effects created by open storage reservoirs in highly urbanized areas. Problems of land acquisition, selection, and availability, are other serious disadvantages of this alternative. A large number of sites are needed and maintenance and personnel requirements would be quite severe. In addition, the existing outfalls would still require measures to eliminate the possibility of dry-weather bypasses and overflows.

Alternative 4a - Deep Tunnel North to Ground Level Storage. This alternative utilizes the elevation differential between most of the service area and the Missouri River to provide energy for conveyance of the overflows to a low-cost storage site. Overflows would be intercepted by drop shafts near the outfalls. The drop shafts would convey overflows to a deep tunnel that would operate as an inverted siphon discharging to a ground level reservoir located across the Missouri River north of the Carter Lake area. The overflows would be pumped

back to the existing interceptor system for treatment at the Missouri River plant. The approximate locations of the various components are shown on Plate 19.

Preliminary geological evaluations, summarized in Appendix C, show that the Mississippian rock formation, consisting of massive limestone and dolomite, is favorable for tunnel construction. Drilling logs for the Omaha area show the elevation of the top of the Mississippian rock formation ranging from +629 to +253 ft, msl. An unlined tunnel could be excavated in these rocks with only moderate amounts of grouting required. Only a few local areas in the tunnel would require structural support. Leakage would probably be minimal, due to the overlying, thick, impermeable shales.

A plot of the surface intersection of the hydraulic gradeline for the 5-year storm is shown on Plate 19. When the design storm occurs, the outfall sewers will surcharge to the gradeline. Combined sewer service connections located at an elevation below the gradeline will have to be rerouted. However, the extent of rerouting would be minimal since the majority of connections below the gradeline are already on separate sanitary sewers. As the deep tunnel fills, the liquid level will backup to the gradeline and the differential head will cause flow to the surface reservoir. This reservoir may be constructed above grade at less cost than an excavated reservoir.

After the storm, the influent gates to the reservoir would be closed and high head pumps would empty the contents

of the deep tunnel and vertical drop shafts to the reservoir. Since the majority of rainfall events will be smaller than the design storm, the system will not always fill sufficiently to feed the reservoir by gravity. The high head pumps would then be used to empty the contents of the deep tunnel to the reservoir. The tunnel would also be sloped for conveyance of overflows to a grit removal facility located at the far end of the tunnel. Larger debris would be removed initially by trashracks at the drop shafts. During extended dry weather periods, the reservoir may be emptied and cleaned.

The ground level reservoir would occupy approximately 200 surface acres. It is estimated that approximately 16 drop shafts would be required varying from 4 ft to 21 ft diameter. The deep tunnel in the southern portion would start at an estimated 15 ft in diameter increasing to 40 ft in diameter at the Missouri River crossing to the storage reservoir.

The major impacts of this alternative are the construction of the deep tunnel system and the construction and operation of a storage reservoir in Iowa. The deep tunnel system would create minimum disruption and modest noise in the vicinity of the tunnel portals and adits. The surface requirements would be limited to small areas with minimum street level disturbance. Disposal of the million cubic yards of rock spoil from the tunnel may be a problem. If the rock is sold for construction use, a substantial storage area probably would be required.

Another environmental impact due to the construction and operation of an aerated storage reservoir in Iowa, would be preempting 200 acres of land currently in agricultural use. The reservoir could be designed and landscaped in such a way as to minimize the visual impact on surrounding areas.

This alternative would entail an estimated initial construction cost of $159 million and total annual cost of operation and maintenance estimated to be $3.4 million. The estimated total present worth cost is $215 million. The individual breakdown of component costs is shown on Plate 14.

This alternative has the advantage of minimizing aesthetic impacts throughout the area, eliminating the need for duplication of facilities at outfall points, providing storage for dry-weather flows during treatment plant failure, and a good potential for public acceptance. The alternative is also economically attractive and competitive.

<u>Alternative 4b - Excavated Storage North - Deep Tunnel South to Ground Storage</u>. This alternative deals with the northern portion of the study area separately from the southern portion. In the northern area, overflows would be diverted to open excavated storage reservoirs in the Carter Lake area with a total capacity of 1350 A-ft. Some additional conveyance of the overflows to the surface storage reservoirs may be required depending on the final location of the reservoirs. The reservoirs would be lined to prevent infiltration. The reservoir, although excavated, would be shallow (10 ft liquid

depth) because of high ground water conditions, thus requiring a large surface area. The overflows in these reservoirs would be pumped to existing interceptor systems for treatment at the upgraded Missouri River plant.

Overflows in the southern portion, beginning with the Burt-Izard outfall, would be handled in a similar manner as described in Alternative 4a. Twelve drop shafts at the outfall sewers would convey the overflows to a deep tunnel and a diked ground level reservoir similar in concept to Alternative 4a, but with only 1900 A-ft capacity. The reservoir would be located across the river from the Missouri River treatment plant and south of Council Bluffs, Iowa. The deep tunnel would vary in size from 18 ft in diameter at Burt-Izard to approximately 27 ft in diameter at Monroe Street. The stored overflows from this reservoir would be pumped back across the river for treatment at the upgraded Missouri River treatment plant. The various components for this alternative are illustrated on Plate 20.

This alternative would involve environmental considerations similar to Alternative 4a. The constructing and operation of storage reservoirs in the Carter Lake area would have a small impact, since the area considered is ecologically disturbed. The ground level storage reservoir, in Iowa across from the Missouri River treatment plant, would require removal from production of about 120 acres of farm land. Since the land surrounding the reservoir site is rural, the visual impact of the reservoir would be minimal.

This alternative would entail an initial construction cost of $102 million and total annual cost of operation and maintenance estimated at $3.4 million. The estimated total present worth cost is $162 million.

This alternative is attractive by cost comparison. In addition, excavated storage reservoirs in the North reduce the emphasis on the hydraulic grade line for gravity flow. This alternative would require less surcharging of existing sewers. Disadvantages of this alternative would be locating and acquiring the necessary sites in the northern area for the surface storage facilities and their unfavorable aesthetic impact.

The facility in Iowa could also provide the necessary storage volume to handle the combined sewer overflows from Council Bluffs. However, these overflows would have to be pumped to the storage reservoir because there is insufficient difference in elevation to allow gravity flow.

Alternative 5a - Deep Tunnel with Mined Storage. This alternative consists of conveyance of combined sewer overflows by deep tunnel to a storage reservoir excavated in the rock by mining methods below the Missouri River treatment plant. High head pumps would elevate the combined sewage from the mined storage reservoir up to the plant for treatment. The components of this alternative are illustrated on Plate 21.

Drop shafts would be located at the existing overflow points and the tunnel would discharge by gravity to the mined

storage reservoir. With the deep reservoir location, head would be available to produce very high velocities in the tunnel, allowing use of smaller diameters than in the tunnel alternatives with ground level storage. Lining of the tunnel would be required with such high velocities (40 ft/sec); therefore, it might be cheaper to construct the tunnel at a shallower elevation in softer rock. The tunnel would convey the overflows to a common vertical drop shaft near the storage facility, located in the Mississippian dolomite formation at 400 to 700 ft depths. The mined storage facility would require a volume of approximately 3000 A-ft.

The impacts of this alternative would stem entirely from the underground construction of the deep tunnel conveyance system and the mined storage reservoir. The deep tunnel system would produce about half a million cubic yards of rock spoil and the storage reservoir would produce six million cubic yards. If utilized in other forms of construction, this rock would have to be stored; if not salable, it would have to be disposed of at a land site.

This alternative would entail an initial construction cost of $252 million and total annual cost of operation and maintenance estimated at $3.5 million. The estimated total present worth cost is $308 million. This estimate includes the cost of construction, operation, maintenance, treatment and land. Individual component costs are summarized on Plate 14.

This alternative has the advantage of excellent potential for public acceptance since all of the facilities would be underground with no visual or aesthetic impact on the environment. The high cost and the need for disposal of the spoil produced during construction are disadvantages.

Alternative 5b - Excavated Storage North-Deep Tunnel to Mined Storage South. The cost of this alternative is reduced, as in Alternative 4b, by dealing separately with the northern and southern areas. This alternative requires two excavated storage reservoirs in the Carter Lake area comparable to those described under Alternative 4b. The overflows, after reservoir retention in the North, would be evacuated to the existing interceptor system for treatment at the upgraded Missouri River treatment plant.

The outfalls in the South, including Burt-Izard, would discharge to vertical drop shafts, deep tunnel, and a mined storage facility. After storage, the overflows would be pumped to the Missouri River treatment plant. The components are illustrated on Plate 22. Approximately 12 drop shafts would be required varying from eight feet to 18 ft in diameter. The deep tunnel would range from a minimum of 10 ft in diameter at the Burt-Izard outfall to 18 ft in diameter at the mined storage reservoir. The mined storage facility would be approximately 2000 A-ft in size.

This alternative would entail an estimated initial construction cost of $175 million. The annual cost for operation

and maintenance is estimated at $3.4 million. The estimated total present worth cost is $233 million. Land costs for the surface reservoirs in the Carter Lake area are estimated at $5.5 million. Cost components are summarized on Plate 14.

This alternative is less costly than the previous alternative that uses deep tunnel and mined storage exclusively. The likelihood of public acceptance for this alternative is not as high because of the use of surface reservoirs in the Carter Lake area.

Alternative 6 - Flow-through Treatment with Storage at Outfalls. This alternative is based on use of facilities for treatment of combined sewer overflows at the outfall points. The systems that have been demonstrated to date consist of initial screening for removal of coarse solids, pressurized dissolved-air flotation for removal of finer solids, chemical addition for coagulation and removal of nutrients, and chlorination for disinfection. Locations of individual treatment facilities are shown schematically on Plate 23.

In the Omaha combined sewer system, the drainage areas are so large that peak flows generated during the design storm cannot economically be handled directly by flow-through treatment units. This alternative, therefore, includes provision of storage at the outfall points for attenuation of the peak flows before pumping to the flow-through treatment units. Individual treatment facilities

would vary from about 1.5 to 110 mgd. The total flow-through treatment capacity required for the study area would be approximately 390 mgd for the 5-year recurrence interval. Buried concrete storage tanks would be evacuated by pumping stations at a rate equivalent to a maximum detention period of 10 days. This treatment process, which is limited to only meeting the variance level of treatment is further described in Appendix D.

The essential impacts of this alternative would occur at the individual outfalls, where the construction would take place. Excavation of underground retention reservoirs at the individual outfalls would produce substantial spoil. Where treatment plants would be visible from residences or parks, imaginative architectural and landscape design should be employed to minimize the visual impact.

This alternative would entail an estimated initial construction cost of $376 million. The annual operating and maintenance costs are estimated at $3.0 million. The estimated total present worth cost, for a variance level of treatment, is $408 million.

This alternative has the disadvantage that it cannot be directly compared with previously described concepts because it is incapable of achieving secondary level treatment. However, treatment to the variance level may be adequate depending on final guidelines and allowance levels for effluent water quality. Other disadvantages include the visual impact

and the multitude of facilities that will have to be operated and maintained.

Alternative 7 - Sewer Separation. This alternative entails separation of existing combined sewers by construction of new sanitary sewers. It was assumed that storm runoff and snow melt water would not require treatment and would be acceptable for discharge to the Missouri River. One way that the existing combined sewer system can be separated would be to install new sanitary sewers inside the existing combined sewers. It is less costly to use this approach as opposed to installation of a new sanitary sewers in separate trenches. In any case, the construction costs are extremely high. In 1970, the combined sewers of about 100 acres in the downstream area were separated near the Leavenworth Street outfall. The costs for separation in the area were approximately $18,000 per acre. Projects elsewhere show an approximate cost for separation of $20,000 per acre served, but they were completed around 1970.[1/] To reflect higher construction costs, a cost of $27,000 per acre was used in this study.

This alternative would entail an estimated initial construction cost of $539 million. The estimate includes necessary costs for construction, with replacement and repairing of existing facilities. This alternative is listed in this study as a point of reference since it is not cost effeftive when compared with the other alternatives. The disruptive effect throughout the urban area and low likelihood of public acceptance also make this alternative undesirable.

1/ Sewer separation cost experience and studies are described in Appendix E.

Alternative A - In-System Attenuation Devices - This alternative would utilize power operated storm flow regulators for storing flows within the pipes of the combined sewer system. This method could also be used as a first stage in the implementation of a complete alternative. This system uses computer-assisted real-time monitoring and remote control to regulate flows in the sewer system so as to minimize overflows.

Inflatable fabric dams or hydraulically operated gates with support equipment that would be housed in an underground vault, are the major components of the control device. The fabric dam does not completely close off the trunk sewer and water can flow over its top. When deflated, the dam closely conforms to the sewer walls and base to afford almost full hydraulic capacity of the pipe. When a rainfall event occurs the fabric dam is fully inflated, causing the level in the sewer to increase, thus utilizing the sewer for in-system storage.

The Minneapolis-St. Paul Sanitary District obtained an EPA grant for demonstrating the feasibility of the above method. The combined sewer monitoring and control system was placed in operation in the spring of 1969. During 27 rain events in 1969 and early 1970, the number of overflows was reduced by 58 percent and the duration by 88 percent. Subsequent analysis of data from all of 1970 showed similar results with reductions in overflow incidence and duration of 52 percent and 84 percent.

It is estimated that this method could reduce the number of overflows to the Missouri River from approximately 50 incidences per year to less than 5. Although this alternative does not meet the complete objectives of the study, it is presented here as a partial solution. The alternative may also be attractive if a lower level of treatment is allowable or as a staging concept in connection with other alternatives previously discussed.

The estimated initial construction cost of this type is $83 million and the annual cost of operation and maintenance is $1.7 million. The estimated present worth for this alternative is $123 million. The estimates include construction, operation, maintenance, and necessary equipment.

Alternative B - Deep Tunnel to the Papillion Creek Sewage Treatment Plant. The abatement of pollution from combined sewer overflows from the Omaha-Missouri River system is only one segment of wastewater management planning in the Omaha metropolitan area. Other water quality management planning activities involve an evaluation of the sewer systems in Omaha and Council Bluffs and the formulation and evaluation of alternative regional wastewater management plans.

This alternative, which could be incorporated into the regional plan, entails deep tunnel conveyance to a mined storage facility near the Papillion Creek Sewage Treatment Plant and is illustrated on Plate 24. No provision are

included for treatment and disposal of the combined sewer overflows. Overflows would be diverted at the outfall points through drop shafts to a deep tunnel continuing beyond the Missouri River Treatment Plant to a mined storage facility near the Papillion Creek Treatment Plant. The deep tunnel would range from eight feet to 34 ft in diameter.

This concept could be used to eliminate either the proposed secondary facility or the entire Missouri River Treatment Plant. The primary effluent from the present Missouri River Plant or raw sewage from the outfall points would be diverted to the deep tunnel for conveyance to the storage facility and eventual treatment. The tunnel itself would also provide storage during the majority of rainfall events.

The estimated initial construction cost would be $207 million. The estimated present worth for this alternative is $240 million. These costs only include the drop tunnel conveyance and mined storage near the Papillion Creek Treatment Plant. No treatment, disposal, or land costs are included.

Alternative C - Flow-through Treatment for "First-Flush". A concept meriting consideration would be to provide partial treatment at the outfall points with screening/dissolved-air flotation units to handle the "first-flush" effect. The peak flow does not generally occur until the "first-flush" effect is completed. Based on the above, treatment units would be

sized accordingly and placed at the outfall points to handle the more concentrated overflows. Subsequent overflows at lower concentration levels, would be allowed to bypass to the river untreated.

The estimated construction cost for the treatment units for this alternative would be $279 million. The estimated present worth of this alternative is $337 million. These costs include equipment, installation, construction, operation, maintenance, and land.

This method would not require additional storage or conveyance in the system. Additional treatment capacity at the Missouri River Plant would not be required. This concept is illustrated on Plate 25.

Selection of Alternative Concepts for Evaluation as Alternative Plans

The most important criterion for comparison of equally performing alternatives is total cost. The subjective qualities of an alternative should also be considered if the alternatives do not perform equally or if the total costs do not differ appreciably. Seven of the twelve alternatives evaluated are close enough in cost to warrant comparison of their subjective qualities. The other five could be eliminated from further evaluation solely on the basis of cost.

The five alternatives that have excessively high costs, greater than $300 million, are buried storage near the overflow points (Alternative 1); deep tunnel to mined storage (Alternative 5a); flow-through treatment at the overflow points (Alternatives 6 and C); and sewer separation (Alternative 7).

The buried storage tanks associated with Alternative 1 would be relatively expensive retention facilities because of the structural requirements necessary to overcome the bouyancy forces of the high water table. Alternative 5a would require construction of a tunnel several hundred feet underground in sound rock to convey the overflows to an underground reservoir constructed by mining methods. The cost of underground mined storage, however is less expensive then the cost of buried storage tanks. Flow-through treatment facilities, associated with Alternatives 6 and C, located near each overflow point are not only expensive, but would provide less than the equivalent of secondary treatment. Alternative 7, which employs sewer separation by construction of separate sanitary sewers inside existing combined sewers, is the second most expensive alternative evaluated, $539 million. In addition to the high cost, this alternative would be the most disruptive of all twelve alternatives evaluated and would not provide treatment for the separate storm runoff as would all other alternatives.

Four alternatives do not perform equally with the other eight. They are the two previously discussed flow-through treatment alternatives (Alternatives 6 and C), in-system

attenuation (Alternative A), and deep tunnel to Papillion Creek (Alternative B). In-system attenuation would be effected by installation of computer operated hydraulic controls to utilize the volume of sewer pipes for storage of runoff. This alternative has the lowest total cost, $123 million, but can only reduce the number of overflows per year from 50 to about 10. Thus, as an alternative for the entire combined sewer area, it would be inadequate in performance. Alternative B would require construction of a deep tunnel, as an extension to the one described for Alternative 5a, to convey overflows to the Papillion Creek treatment plant site. This alternative was formulated so that it could be further evaluated as part of the regional wastewater management study for the Omaha-Council Bluffs area; and consequently, it does not include costs for treatment. This alternative will be further evaluated as a partial alternative plan.

Five equally performing alternatives, which are the lowest cost alternatives, range in total present worth cost from $161 million to $233 million. These alternatives, from lowest to highest total cost, are deep tunnel south and excavated open surface storage in the North (Alternative 4b); upstream retention (Alternative 3); diked storage along the levee (Alternative 2); deep tunnel north to ground level storage (Alternative 4a); and excavated storage in the North with deep tunnel to mined storage in the South (Alternative 5b).

Alternative 4b, the lowest total cost of these five alternatives, would cause minimal disruption during construction.

Aesthetically and environmentally, it compares very favorably with all other alternatives. The ground level reservoirs would have a low profile to minimize visual impact and the excavated open surface reservoirs would be located in non-residential areas. Provisions for adequate aeration would prevent occurrences of odor problems.

Alternative 3 would utilize excavated, open, surface reservoirs and buried tanks located throughout the sewer system. Although this alternative is relatively low in total cost, it would be very disruptive during construction. Several retention reservoirs would be located in residential areas where, aesthetically and environmentally, they would be unattractive. Because there would be about 30 of these retention structures throughout the system, maintenance of these facilities might be a problem.

Alternative 2 provides for storage of the combined sewer overflow in elongated reservoirs along the river side of the levee. All construction would take place in the undeveloped areas along the river instead of in the urbanized area. This alternative would be as aesthetically attractive as Alternative 4b. About 40 percent of the total cost, which is for pre-storage conveyance to the reservoirs, could be substantially reduced if further evaluation indicates that not all overflows must be pumped to the reservoirs.

Alternative 4a is similar to 4b except that all overflow is conveyed by a deep tunnel to one large ground level reservoir north of Council Bluffs. Aesthetically this

alternative is more attractive than Alternative 4b because there are no excavated, open, surface reservoirs. The ground level reservoir would be located in primarily agricultural land, away from any residential areas. The reservoir would have a low profile and minimal visual impact. Disruption during construction would also be minimal.

Alternative 5b, which is a modification of 4b, would utilize mined storage in place of the ground level reservoir south of Council Bluffs. This alternative has the highest cost of the five, and its only advantage is that the mined storage is not visible to the public. Mined storage, as discussed in Alternative 5a, is relatively expensive and should be considered only if further evaluation indicates serious problems with implementing alternatives with storage near ground level.

The twelve alternatives were formulated and costed for the same combined sewer area. The comparison and selection of alternatives for further evaluation should not, however, eliminate a concept that may be attractive for an individual service area even though it appears unattractive for the entire area. For this reason the concept of upstream retention, sewer separation, and in-system attenuation should be considered for individual service areas during refinement of the most attractive areawide alternatives.

The selected alternatives for further refinement are:

- Alternative 2 - Diked storage along the levee;
- Alternative 4a - Deep tunnel north to ground level storage;
- Alternative 4b - Deep tunnel south and open surface storage in the North;
- Alternative 5a - Deep tunnel to mined storage; and
- Alternative B - Deep tunnel to Papillion Creek.

Deep tunnel to mined storage will be evaluated further as a substitute to the surface storage, which may be impossible to implement. The extension of this concept to Papillion Creek will be refined for the purpose of providing sufficient input data for the regional study. Also, the concepts of upstream retention, sewer separation, and in-system attenuation will be considered for individual service areas as part of the evaluation of alternative plans.

The planning strategy for Phase I of this study involved formulation, selection, and preliminary evaluation of alternative concepts concluding with the recommended concepts to be further refined as alternative plans. The costs generated in this chapter and retermed on various plates are preliminary and will be refined for those alternative concepts selected for further study.

The evaluation and refinement of alternative plans constitute Phass II of the study and will be presented in the next chapter of this report. The refinement of alternatives may require adjustments in formulation resulting in corresponding changes in cost. Constitution costs for the alternative concepts evaluated in this chapter would therefore, not be identical to the costs developed for alternative plans in the next chapter.

Chapter VII

REFINED STUDY OF THE FIVE MOST ATTRACTIVE ALTERNATIVES

Through a process of successive steps of screening and refinement, five concepts were selected for formulation of alternative plans for combined sewer pollution abatement. This selection was made jointly by the study staff and a coordination committee composed of all concerned agencies. The factors considered in formulating the alternative plans and descriptions and evaluations of the alternative plans are presented in this chapter.

Plan Formulation

A design recurrence interval of five years was selected as a reasonable basis for comparing the many alternative concepts, as discussed in Chapter VI. However, a specific design recurrence interval has not been established by law as yet, and may be dependent on other factors in the area-wide water quality management plan. Therefore, each alternative plan has been sized and cost estimates prepared for a range of recurrence intervals from one year to 10 years.

Storage Volume

All five alternative plans include provision for storage to retain the overflows before pumping at a controlled rate to a treatment facility. The size of each storage facility is dependent on the recurrence interval, the treatment rate, and the area served. For a particular recurrence interval, the

required storage volume will decrease as the treatment rate is increased.

A storage volume equal to the 48-hour overflow and a treatment rate that will empty the basin in 20 days will provide a reasonable combination to meet design conditions. The minimum required storage volume would be equal to the 24-hour overflow volume because the interceptor system and sewage treatment facilities would be taxed to their maximum capacity during the first 24 hours. Treatment rates assumed for the alternative plans range from 37 mgd to 71 mgd depending on design recurrence interval. The absolute minimum treatment rate would be 12 mgd, equal to the equivalent annual average rate of overflow. The economic optimum between storage volume and treatment rate will depend upon costs for storage and treatment and constraints imposed by the existing interceptor system capacity. However, it is expected that the optimum treatment rate will be greater than the minimum rate of 12 mgd.

Treatment Processes

All of the alternative plans have been formulated on the basis of storage of the combined sewer overflows and treatment at a controlled rate. The flow to the treatment process was assumed to be intermittently pumped at a constant rate. Not all conventional treatment processes can adapt to these conditions. The activated sludge process usually requires a continuous flow input and must be modified to be a practical process for treatment of combined

sewer overflows. For the comparison of alternative concepts, it was assumed that part of the stored overflows could be pumped to the proposed activated sludge treatment plant at the Missouri River treatment plant during low flow periods.

The amount of stored overflow which could be treated at an upgraded Missouri River plant depends upon the design capacity of the plant and the diurnal variation in flow rate. At this time, the dry weather flow averages 30 mgd over a 24 hour period but decreases to 20 mgd during the night. The expected variation in flow and the design capacity of the proposed activated sludge process, however, are not available at present. Therefore, it was assumed for the development of alternative plans that there would be no available capacity at the upgraded Missouri River plant to treat the stored overflows during low flow periods and that all treatment capacity for stored overflow must be provided in a separate treatment plant. This conservative assumption does not affect the comparison of alternative plans.

Treatment of combined sewer overflow was assumed to be provided by a contact-stabilization process or by an aerated lagoon system. The modified contact-stabilization process was the least cost process evaluated for treatment of the stored overflows. Based on the experiences at the Kenosha,

Wisconsin STP, construction costs were estimated at $0.13 million per mgd of treatment plant capacity. Other processes investigated were activated sludge with an estimated cost of $0.58 million per mgd, physical-chemical treatment at $0.90 million per mgd, and aerated lagoons at $0.33 million per mgd. The aerated lagoon system is at low cost process but requires large usage of land accompanied by potential aesthetic and public acceptance problems.

A modification of the contract stabilization activated sludge process, called the bio-treat process, consists of a wet-weather flow facility operating in conjunction with the dry-weather treatment plant. During dry-weather periods a fresh supply of activated sludge solids is maintained in an aerated stabilization tank through the periodic wasting of sludge from the dry-weather treatment plant. When the overflows are pumped from the storage reservoirs they are directed to a contact basin along with solids from the stabilization tank. Following a short period of aeration and mixing in the contact basin, the flows proceed to the wet-weather final clarifier where the settable solids are removed and returned to the process. The effluent would be disinfected and discharged to the Missouri River. If advanced waste treatment is required for further BOD, suspended solids, or nutrient removal, then additional treatment components such as filtration, activated carbon, or ion-exchange would be added. During dry-weather periods the

wet-weather clarification equipment can also be used to supplement the existing plant. The U.S. EPA has provided a demonstration grant for the City of Kenosha, Wisconsin, to test the effectiveness of the modified contact stabilization process to treat combined sewer overflows.[1/] The dry-weather capacity at the Kenosha plant is 18 mgd. The wet-weather capacity of the modified biological system is 20 mgd. The wet-weather plant has been able to achieve treatment quality equivalent to conventional secondary treatment. Also, the dry weather-effluent at Kenosha has averaged less than 15 mg/l of suspended solids and BOD with the additional capacity. The rapid start-up characteristics of the system assure treatment of the stored overflows as they are pumped to the facility. The use of a standby type facility is well suited to the intermittent flow and variable characteristics of combined sewer overflows.

The aerated lagoon system followed by a storage reservoir is also readily adaptable to intermittent inflow conditions because of the relatively long detention times employed provided land is available and public acceptance can be obtained. There have been few reports on the

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1/ Agnew, et. al., "Biological Absorption System for the Treatment of Combined Sewer Overflow", presented at the 46 Annual WPCF Conference, Oct. 1973.

use of this process to treat stored combined sewer overflow, but it has all the necessary requirement to do so.

Operating the storage reservoir as an aerated lagoon system is an approach that could eliminate the need for further treatment process. However, there is no available data on such systems to provide the minimum necessary level 1 treatment of combined sewer overflows. Determining the design requirements and predicting the behavior of such a system is a complex problem that is beyond the scope of this study.

Additional BOD and suspended solids removal could be effected with filtration facilities, and phosphorous removal could be effected with the use of alum addition, coagulation and settling. To meet the background water quality standards, more advanced methods of treatment such as ion-exchange or reverse osmosis may be required. The removal of ammonia and nitrogen after the bio-treat process could be effected by using ammonia stripping towers. This type of ammonia removal system appears to be more attractive than the biological nitrification system because of the intermittent nature of the flow to the treatment process. Freezing of towers during winter months, the major problem associated with ammonia stripping towers, is not anticipated to be a major problem because the use of this facility would be minimal during the winter months. All of the overflows could be treated during the warmer months of the year.

There could be significant reuse potential of the combined sewer overflows if advanced treatment were provided to meet background water quality levels. Potential re-use applications would include land application as irrigation water, industrial applications for cooling and process water, and development of lakes and ponds along the river front for recreational purposes.

Measure of system performance

The performance of alternative plans was evaluated on the basis of annual overflow volume captured. Based upon 20-years of rainfall record (1951 through 1970), the annual average untreated overflow volume from the present combined sewer system is estimated to be 15,300 A-ft. A combined sewer overflow abatement facility designed to capture and treat an event with a 1-year recurrence interval will reduce the annual untreated overflow volume to an estimated 865 A-ft. As shown in Table VII-1, the design of larger facilities will not substantially decrease the volume of untreated overflow.

The BOD_5 and suspended solids reductions associated with each design were estimated by assuming average concentrations in the untreated combined sewer overflow of 100 mg/l BOD_5 and 250 mg/l suspended solids. When sizing for larger recurrence intervals, the percent removal of

Table VII-1

ANNUAL UNTREATED OVERFLOW CHARACTERISTICS

SYSTEM DESIGN	Number of Untreated Overflow events		Volume of Untreated Overflow		Treatment Level[1/]	BOD_5 Discharge in Combined Sewer Overflow		Suspended Solids Discharge in Combined Sewer Overflow	
	No.	Percent Reduction	Acre-feet	Percent Reduction		Million Pounds	Percent Reduction	Million Pounds	Percent Reduction
Existing	50	0	15,300	0		4.00	0	8.0	0
1-yr Recurrence Interval	1	98	865	94.3	I	1.41	64.8	1.77	77.9
					II	0.82	79.4	1.37	82.8
					III	0.43	89.2	0.67	91.7
2-yr Recurrence Interval	0.5	99	475	96.9	I	1.34	66.5	1.53	80.9
					II	0.73	81.7	1.13	85.9
					III	0.33	91.7	0.40	94.9
5-yr Recurrence Interval	0.2	99.6	180	98.8	I	1.28	68.0	1.35	83.1
					II	0.66	83.4	0.94	88.2
					III	0.25	93.6	0.20	97.5
10-yr Recurrence Interval	0.1	99.8	75	99.2	I	1.26	68.5	1.29	83.4
					II	0.64	83.9	0.88	89.0
					III	0.23	94.3	0.14	98.3

1/ Level I is secondary treatment
I l II is advanced treatment
L l III is background water quality

BOD_5 and suspended solids approaches an upper limit that depends upon the level of treatment provided. These percentages do not change appreciably, as shown in Table VII-1.

Table VII-1 may be somewhat misleading by indicating what appear to be modest removals of BOD_5 and suspended solids for Level 1 or secondary treatment. The pollutant load to the river, however, would be discharged at a reduced rate after attenuation through the system. The elimination of shock loadings might be as beneficial to the Missouri River as treatment of the overflow. This observation would have to be evaluated by a waste assimilation study of the Missouri River.

The performance under overload conditions will be approximately the same for each alternaive plan. If an alternative is designed for Level 1 treatment and a 5-year recurrence interval it will produce a minimum BOD_5 of 30 mg/l based on an influent BOD_5 concentration of 100 mg/l for all events of 5-years or less. If the 10-year event should occur the effluent would increase to 60 mg/l, for the 25 year event 80 mg/l, and for the 50-year event 90 mg/l. The overload performance curves are further summarized on Plate 13.

Special Considerations for Specific Service Areas

Each alternative plan employs a basic concept that appears attractive with respect to the entire combined sewer

service area. However, further examination of an alternative plan may indicate that previously eliminated concepts may be more attractive for a particular sewer service area. Thus, concepts such as sewer separation, upstream retention, and in-system attenuation could be utilized as local elements in the formulation of an alternative plan. The following sections describe these elements, indicate service area characteristics that make each element economically attractive, and identify possible locations for their application in the study area.

Sewer Separation

Sewer separation is the elimination of combined sewer overflows by separation of storm water runoff and sanitary sewage. Instead of storm water runoff and sanitary sewage being combined into a single sewer system, the flows are directed into two separate collection systems.

Sewer separation to eliminate combined sewer overflows in Omaha would be practical in areas where the existing combined sewers are adequately sized to carry storm runoff without causing local flooding problems. In most cases, conversion of the existing combined sewers to carry only sanitary sewage would allow heavy deposition of solids in the sewers and cause attendant odor and flow problems. The solids would settle out because the velocity of the sewage in the oversized sewers would not be sufficient to

maintain the solids in suspension. Consequently, sewer separation to eliminate combined sewer overflows in Omaha would normally require installation of new separate sanitary sewers, rather than new separate storm sewers. Storm runoff would continue to drain to the existing collection system and discharge directly to the Missouri River. Sanitary sewage would be collected from the new sanitary sewers and flow to the North and South Omaha interceptors.

Most of the older structures in the combined sewer service area have combined house or building sewers. Many of the roof drains (downspouts) and footing drains are connected to the main building sewer. Separation of sewers may require not only new sewers under the streets, but also plumbing modifications in each structure to eliminate water entering from drains to the separate sanitary sewers.

Some sewer separation has been undertaken in Omaha to alleviate local flooding problems by installing a new separate storm sewer above the existing combined sewer. However, the separated sanitary sewage and storm runoff are mixed together outside the flooding problem area. Total sewer separation in these areas therefore could serve two purposes, reduce flooding and eliminate combined sewer overflows, and thus, reduce the cost attributable to the elimination of combined sewer overflows.

Characteristics of a sewer service that lead to high costs of sewer separation include high density housing, and uniform distribution of structures.[1/] Another factor is the distance from the Missouri River. Long distances indicate that a substantial cost for conveyance of storm water or sanitary sewage will be incurred.

Sewer separation should be considered as an element of an alternative plan wherever the particular characteristics of a service area economically favor sewer separation. Sewer separation may be an attractive element in an overall plan for areas adjacent to the interceptor system, for low-lying areas that are difficult to pick up by deep tunnel conveyance, and for areas that require expensive conveyance to diked storage facilities.

Separation of combined sewers effect only minimal economic savings if done after an overall plan for combined sewer pollution abatement is implemented. Should the construction of a combined sewer pollution abatement facility precede the construction of a Riverfront Development Project, then there would be little advantage in separating the sewer system as part of the project. On the other hand, if a redevelopment project in a combined

1/ Sewer separation and cost expenses are discussed in Appendix E.

sewer service area is initiated prior to the construction of a combined sewer pollution abatement facility, then sewer separation for specific areas should be considered as a method of reducing the overall cost.

Upstream Retention

Local flooding problems can also be alleviated by upstream retention rather than sewer separation. This alternative element merits consideration because of its potential dual function. Areas that have experienced local flooding problems are shown in Plate 26.

Upstream retention involves the diversion of storm runoff and sewage from combined sewers into retention basins. These retention basins could be strategically located to reduce surcharging of sewers in problem areas and also to reduce the flow rate and volume of combined sewage at the overflow points. Thus, upstream retention can reduce not only the size of storage facilities, but also the size of pre-storage conveyance facilities.

The cost of upstream retention facilities depends upon the type of storage facility constructed. Buried or covered storage tanks located under parks, streets, parking lots, and other open space areas would not preempt the existing land use, but would cost more than open storage

reservoirs. If the ground water table is high, bouyancy forces will greatly increase the cost of covered storage.

The cost of upstream retention will also be a function of size and land requirements. In locations which do not have a ground water problem, the estimated unit cost of covered storage ranges from $50,000 per A-ft for a 10 A-ft storage facility to $27,500 per A-ft for a 100 A-ft storage facility. The major land cost for covered storage facilities will be for easements.

Open storage reservoirs for combined sewage would be unattractive in residential neighborhoods or any developed urban areas. They would have to be fenced off from the public; and the land would be committed to a single purpose. To allow gravity flow of combined sewage into the open reservoir, the reservoir would have to be excavated so that the design water surface elevation was below the invert elevation of the tributary sewers. In most of the service areas, this would require excavation of an additional five to 10 feet below grade. Consequently, the cost of open storage in these areas would be about $14,000 per A-ft. This cost does not include the cost of land acquisition. Land costs for open storage reservoirs will depend upon the location and the areal extent required for the reservoir.

Buried storage reservoirs allow more flexibility in land use than do open reservoirs. However, this flexibility is gained with a higher construction cost. Because these storage reservoirs are located underground there are almost no constraints on their geographic location within the sewer system.

This rather expensive measure for storing combined sewage can be economically utilized in areas where it will effectively reduce the peak flow rate at the overflow point. The attenuation of peak flows by upstream retention can substantially reduce the size of pre-storage conveyance facilities. With the peak flow reduced 20 percent by attenuation, the cost of upstream retention must be less than $30,000 per A-ft of storage in order to effect a cost-savings in the construction of a deep tunnel conveyance system. For planning purposes, the specific sites of upstream retention are not identified as part of the alternative plans. However, as an example, upstream retention in the Grace Street service area might be an attractive element to reduce the cost of deep tunnel conveyance in Alternative 4a.

<u>In-System Attenuation</u>

In-system attenuation makes use of the unused storage capacity within the existing sewerage system for the purpose of reducing the frequency and volume of combined sewer overflows. This type of system utilizes power-operated storm flow regulators for storing flows within

the sewer pipes. As an element of alternative plan, this system could be used to reduce the pre-storage conveyance capacities and the storage capacities of holding reservoirs.

A continuously adjustable combined sewer regulator is the major field device involved in this system. The major components of the modified regulator are an inflatable fabric dam, a hydraulically operated gate, and support equipment housed in an underground vault. The fabric dam will not completely close off the trunk sewer so that water can flow over its top. When deflated, the dam closely conforms to the sewer walls and base to afford almost full hydraulic capacity of the pipe. When a rainfall event occurs, the fabric dam is fully inflated, causing the level in the sewer to increase, and thus, utilizing the sewer volume for in-system storage.

Operation of this type of system must be based upon information of the conditions in the sewer system. A system of remote monitoring devices, such as rain gauges, water level sensors, gate and dam sensing devices and river quality sensors, provides the necessary information. Depth of flow at each regulator is measured at two locations, upstream from the diversion dam and downstream from the regulator gate position. Dam inflation pressure is also monitored.

The performance of the in-system attenuation process is largely dependent on the imperviousness of the tributary area, the time elapsed since the previous rain, the intensity or length of storm, and the level of the river.

Trunk sewers, with diameters greater than five feet, are located near each of the outfalls. In the study area, the grades of these sewers, both the rectangular boxes and the cylinders, are relatively flat. Therefore, these large sewers could be utilized for storage for the in-system attenuation process.

The costs of establishing the in-system attenuation is on the order of $3,000/acre served. The operation and maintenance cost for this system has been estimated at $300,000/yr for the system constructed in Cleveland.[1/]

In-system attenuation has the following advantages: 1.) it is an effective way of preventing small storm run-off from being discharged into a waterway; 2.) during large storms "the first flush" can be captured and held for treatment after the storm has ended; 3.) the regulator system can be operated both manually and automatically; 4.) gates are not required to maintain the regulator positions;

1/ Pew, et. al., "Data Acquisition and Combined Sewer Controls in Cleveland", Journal, Water Pollution Control Federation, Vol. 45, No. 11, Nov. 1973.

5.) the inflatable dam will deflate automatically upon interruption of power so that flooding damage cannot result from loss of control.

However, operation of this type of system can have several disadvantages: 1.) malfunctioning of regulator devices due to fouling by debris and sand found in the combined sewer system; 2.) sediment build-up due to the velocity being reduced below two ft/sec during the time of storm capture; 3.) problems associated with the construction and start-up of the in-system process; 4.) and inability of the system to handle a large rainfall event; 5.) and the possibility of localized flooding.

The total storage volume available in the combined sewer system in Omaha is estimated to be 300 A-ft. Based on a cost of $3000 per acre of service area (the reported construction cost for the Detroit system), the cost of storage is about $220,000 per A-ft. of retained overflow. This cost makes this system prohibitively expensive for consideration as an element of an alternative plan.

Alternative Plans

The selected alternatives evaluated for Phase II of the project study are listed below.

1. Diked Storage Along Levee.
2. Deep Tunnel North to Ground Level Storage.
3. Excavated Storage North - Deep Tunnel South to Ground Storage.
4. Deep Tunnel With Mined Storage.
5. Deep Tunnel to the Papillion Creek S.T.P.

Alternative plans were evaluated on the basis of component costs and total present worth costs for different recurrence interval storms and levels of treatment. The individual component costs of the alternative plans studied for the 5-year recurrence interval are shown on Plate 27. Total present worth costs for all alternative plans for these levels of treatment and four recurrence intervals are shown on Plate 28. An environmental assessment summary table on Plate 29 shows the major impact factors for each alternative plan. In addition, a summary of projected implementation dates for key activities is shown on Plate 30 for the five alternative plans.

Alternative 2 - Diked Storage Along Levee

The primary function of all the alternative plans developed is to capture, retain, treat, and discharge to the Missouri River combined sewer overflows that meet the required

effluent standards. For this alternative plan, the existing sewers would be utilized as the storm flow conveyance system. The existing flood control levee would be used as one side of diked reservoirs constructed along the riverfront to capture and retain overflows. The overflows that are presently discharged to the Missouri River would be diverted by control structures at the outfalls.

This alternative plan would utilize five diked storage reservoirs, located near the Minne Lusa, Grace Street, Leavenworth, South Omaha, and Monroe Street outfalls. In addition, the plan would require diversion structures at the outfalls, conveyance systems from selected outfalls to the reservoirs, floating aerators in the reservoirs, pumping stations to pump out the stored overflows, and treatment facilities. The locations of the major components required for this alternative plan are shown on Plate 31.

Storage Reservoirs

The five storage reservoirs will all have similar designs, and in all cases utilize a portion of the existing levee as one side of the reservoir. The reservoir bottom would be approximately at the invert elevation of the existing outfalls. A typical cross section and drawing of a diked reservoir is shown on Plate 32.

The reservoirs would be watertight to prevent contaminated leakage into the surrounding ground water and the Missouri River as shown in the typical cross section. This

would be accomplished by the use of a three foot layer of clay followed by 12 inches of riprap upon nine inches of bedding on the reservoir sides. Side slopes of the reservoir would be 2.5 horizontal to 1.0 vertical. Both sides of the reservoir would have a 15 to 20-foot top width that could be converted to a river front scenic roadway. Above the riprapped surface, the dikes would be protected from erosion by a grass cover. To protect against uplift, the bottom of the reservoir would be above the ground water elevation, except for small areas directly beneath the aerators to provide adequate pumping volume for the aerators at the minimum liquid level.

Reservoir dimensions were chosen to utilize most of the available land between the existing levee and the riverbank. This allows for an increase in the reservoir capacity at a later date by increasing the height of the dikes. Thus the estimated maximum operating depth for each reservoir varies with the recurrence interval. The only exception to the above was the Grace Street reservoir, where the maximum operating depth was fixed at 10 feet for any design event in order to minimize surcharging of the main sewer serving this relatively flat area. More service connections would be affected by surcharging the outfall sewer in this area, as opposed to service areas located on the bluffs and well above the reservoirs.

The Minne-Lusa reservoir would serve the Minne-Lusa and Mormon Street service areas. The reservoir would be

approximately 1,300 feet wide and 1,700 feet long, covering a surface area of 50 acres. The maximum operating liquid depth in the reservoir would be a function of the magnitude of the design storm, varying from approximately 14 feet to 26 feet. The Minne-Lusa reservoir will require 25 to 30 floating aerators, approximately 100 hp each, to maintain sufficient oxygen dispersion to prevent odors.

The Grace Street reservoir would be constructed around a portion of the existing Grace Street Channel, south of Abbot Drive, and would serve the Grace Street and Burt-Izard outfalls. The Grace Street reservoir would be approximately 1000 feet wide and vary in length from 2,200 feet to 4,700 feet depending on the selected recurrence interval. Again, 25 to 30 aerators would be located throughout the reservoir.

The Leavenworth reservoir would be a relatively narrow channel reservoir (300 ft), extending from the Pierce St. outfall to the I-80 highway overpass. This reservoir would also serve the outfalls along the floodwall from just south of Burt-Izard to Leavenworth. Overflows from these outfalls would be diverted by a conduit constructed adjacent to the existing floodwall and running parallel to the river. The reservoir would cover a surface area of approximately 41 acres, and require approximately 20 aerators on 300-foot centers along the 6,000-foot length. Maximum operating depths would vary from 9 feet to 17 feet.

The South Omaha reservoir would be similar to the Leavenworth reservoir, extending from south of the I-80

overpass to the Missouri outfall. The reservoir area, estimated at 43 acres, would serve the five outfalls from I-80 to the South Omaha outfall, and would require approximately 20 aerators.

The Monroe Street reservoir would be the smallest reservoir, about 18 acres, and would require approximately 10 aerators. This reservoir would be located on the riverbank near Mandan Park, and would be 300 feet wide and about 2600 feet long. A summary of dimensions for all reservoirs is shown in Table VII-2.

The existing levee would provide sufficient reservoir depth to contain the 1-year recurrence interval storm. For larger design storms the levee height would have to be increased by as much as 10 feet in order to contain the 10-year storm. In earlier discussions it was assumed that the levee height would have to be increased throughout the Omaha area due to the decrease in the floodway width caused by the encroachment of the reservoir channels in the Missouri River flood plain. An investigation by the Omaha District Corps of Engineers has determined that the encroachment would only increase the flood stage by approximately three inches. This increase is equivalent to a reduction in flood carrying capacity of about 3,000 cfs. It was concluded that it would be more prudent to accept the slightly reduced degree of protection rather than raise all the levees in the area.

Table VII-2

DIMENSIONS FOR

DIKED STORAGE RESERVOIRS

(Alternative 2)

Dimension	Recurrence Interval	Storage Reservoirs				
		Minne Lusa	Grace St.	Leavenworth	So. Omaha	Monroe
Surface Area, acres	1	50.7	60.4	41.3	43.4	18
	2	50.7	76.3	41.3	43.4	18
	5	50.7	99.2	41.3	43.4	18
	10	50.7	116.0	41.3	43.4	18
Length x Width, feet	1	1700 x 1300	2200 x 1000	6000 x 300	6300 x 300	2600 x 300
	2	1700 x 1300	2900 x 1000	6000 x 300	6300 x 300	2600 x 300
	5	1700 x 1300	3900 x 1000	6000 x 300	6300 x 300	2600 x 300
	10	1700 x 1300	4700 x 1000	6000 x 300	6300 x 300	2600 x 300
Maximum Liquid Depth, feet	1	14	10	9	9	13
	2	17	10	11	11	17
	5	22	10	14	14	23
	10	26	10	17	17	26
Estimated Reservoir floor Elevation	1	980	990	970	970	975
	2	980	990	970	970	975
	5	980	990	970	970	975
	10	980	990	970	970	975

Control Structures and Conveyance Systems

The control structures at the outfalls would divert flows that exceed the existing interceptor capacity into the diked reservoirs, or to a short conveyance conduit to the nearest reservoir. It is estimated that an additional two to three miles of gravity-flow conveyance conduit, from 6 to 20 feet in diameter and designed to flow at a maximum velocity of 20 feet per second, would be required to divert flows from the outfalls to the reservoir channels. A conduit would be employed at the northern reservoir from the Mormon Street outfall to the Minne-Lusa reservoir. At the Grace Street reservoir, the Burt-Izard outfall would be diverted by a conduit running north to the reservoir. The series of outfalls from Douglas to Leavenworth would be interconnected with a conduit paralleling the floodwall and flowing south to the Leavenworth reservoir. The conduits would flow partially full during minor storms, but would have adequate slope to maintain solids-cleaning velocities. Costs for the conduits are shown on the summary cost table. Additional force main conveyances would be required from the new pump stations at each reservoir to the interceptor sewer to pump out the reservoirs.

Pumping Stations

Individual pumping stations will have to be constructed near each of the five reservoirs. The stations would serve a dual purpose of emptying the surcharged sewers and pumping

out the reservoir. Some surcharging of the trunk sewers leading to the outfalls is expected due to the elevation of the ground-level reservoirs. In order to re-establish normal, dry-weather flow conditions at the outfalls as soon as possible, the surcharged sewers would be pumped out into the reservoirs by the pumping stations. The pumping stations would also empty the reservoirs into the interceptor sewer after normal flow conditions have been re-established. It is estimated that the surcharge may extend up to 2000 ft. from the outfall back into the system for the 1-year design sotrm. For a 10-year design storm, the surcharge may extend up to one mile from the existing outfall.

Treatment Facility

The cost estimates given include treatment facilities to achieve secondary treatment before discharge to the river. Cost increases for higher levels of treatment would be similar to this alternative plan as for the previously discussed alternative concepts. With the existing conveyance system and expanded Missouri River treatment plant, a modified biological contact process would be the most economical process for achieving acceptable treatment levels. The process is described earlier in this chapter.

System Operation and Performance

Overflow from storms that produce flow less than the design recurrence interval but greater than the capacity of

the interceptor sewer, would be diverted at the combined sewer outfalls to the nearest reservoir. The overflows would be stored and aerated until the reservoir is pumped out to the treatment facility. It is expected that all overflows will be pumped out within a maximum of 20 days.

During a storm greater than the design recurrence interval, the system would operate in the same manner, until the reservoir reaches its maximum operating depth. At this point additional overflow would spill to the Missouri River.

The bottoms of the reservoirs are at approximately the invert elevation of the outfalls, and the outfall sewers will therefore surcharge to about the same elevation as the liquid level of the reservoirs. The maximum surcharge at any outfall sewer would be limited by the maximum operating height of the reservoir, as determined by the design storm. With the exception of Grace Street, the bluffs limit the extent of the surcharge to a relatively small area near the river. For the lower design recurrence intervals most of the existing connections to the sewer would be at an elevation that would avoid flooding problems. For larger design recurrence intervals, say 10 years, a minor number of connections may have to be re-routed to new pumping stations equipped with high head discharge pumps. In either case, the outfall sewers may fill but not be highly pressurized. With some rehabilitation the existing sewers should be adequate to handle this moderate surcharge. At Grace Street

the land is relatively flat and surcharging would be more extensive if the reservoir operating level exceeds an elevation of 1000 feet. For this reason, the maximum operating height would be limited to 10 feet. The reservoir area would be increased to accommodate the larger recurrence intervals.

After the storm subsides, a diversion gate would close and the sewers would be pumped out by a pumping station located at each of the five reservoirs. It is estimated that for the 10-year recurrence interval event the sewers would be pumped out in 12 hours or less. This will insure that the combined sewer functions normally during any additional storms.

A minimum liquid cover during wet weather months of at least three feet of water would be maintained in the reservoir, with the aerators operating to prevent undesirable odor conditions from forming. In late Fall (probably in November) floating dredges would pump the accumulated sludge and grit, estimated by volume to be three to six inches per year, into river barges for transport and disposal by incineration at the Missouri River treatment plant. After the sludge has been removed the reservoir would be emptied, and flushed with river water if necessary. During the winter months the aerators would be moored in an off position on concrete pads on the bottom of the reservoir.

When the existing pumping stations or grit facilities are out of operation the reservoirs would provide a means

of capturing bypasses presently discharged to the Missouri River. Debris would be removed by trash racks at the existing outfall points. In some cases additional facilities would be required to handle debris before the inlet of the reservoirs.

All of the alternative plans refined in this chapter utilize storage reservoirs that would provide capability for emergency storage of raw sweage should there be a failure of the sewage treatment plant.

System Costs

The component costs for conveyance, diversion structures, storage reservoirs, aerators, treatment facilities, and pumping stations are shown on Plate 27 for the various recurrence intervals. The total construction cost, along with total annual operation and maintenance costs are also given on Plate 27. The construction cost is estimated at $71.1 million for the 5-year recurrence interval. Costs are based on the ENR index in June, 1974. The project life used is 50 years with an interest rate of 7%. All project components are assumed to have a life of 50 years, except for the electrical, pumping, aerators, and related equipment items at 25 years. With the system designed to store and treat sufficient volume for the 1-year recurrence interval, the cost would decrease to approximately $47.2 million or

about $24 million less than the cost for the 5-year recurrence interval design.

The construction costs include treatment facilities to meet Level 1 or secondary treatment. If a higher degree of treatment is desired then incremental costs can be added as required. As described earlier, costs are shown on Plate 8 for the various treatment levels.

Special Considerations

Use of in-system attenuation, sewer separation, and upstream retention were considered for individual service areas to aid refinement and evaluation of this alternative plan. Service areas were analyzed to determine the expected benefits for each of the above elements. While sewer separation is generally expensive and difficult to implement, in the case of this particular alternative plan it may be desirable for very small service areas served by existing combined sewers along the floodwall in the downtown and Riverfront New Town-in-town areas. Sewer separation could also be considered for the Mormon Street area as opposed to providing separate conduit conveyance for approximately two miles from the Mormon Street outfall to the Minne Lusa reservoir.

Since all of the overflows would be transmitted through the existing sewers to the riverfront reservoirs without

the use of the additional conveyance (except for the few isolated outfalls as previously described), there would be no benefit derived from attenuating the peaks through the use of in-system attenuation devices. Therefore, no additional consideration was given to utilizing this element in the evaluation of this alternative plan. Similarly, upstream retention basins would not have an advantage for this alternative plan.

Associated Effects

The evaluation of alternative plans requires the consideration of several special effects. These effects are discussed below for diked storage along the riverfront.

Energy. The major components utilizing electric power as a source of energy in this alternative plan are the floating aerators in the reservoirs. The annual cost of power for operation of the aerators is estimated at $1.7 million per year.

Manpower. The estimated number of personnel required for operation and maintenance of the storage facilities, pumping stations, diversion structures, sludge facilities, and treatment plant would be 20 full time employees. This estimate is only for the facilities required for the combined sewer overflow abatement alternative plan and does not include personnel required to operate any existing wastewater collection and treatment systems. Requirements

would be higher during periods with fluctuating flows due to intermittent wet weather and dry weather periods. It does not appear that the system would be susceptible to labor strikes and shutdowns since minimum personnel could operate the system for relatively long periods. The system could also be automated to include operational sequencing of aerators.

Resources. This alternative plan does not have the same potential to conserve existing resources as other plans. Most of the land that would be utilized by the storage reservoirs is vacant at this time. Depending on the recurrence interval for sizing, 200 to 300 acres of riverfront land would be used. Implementation of this alternative plan would constitute a reduction in land resources and a preemption of land which could be used for other purposes, such as the Riverfront Development Program.

Multi-use Potential. The concept of an aerated channel, 200 to 300 feet wide, along the riverfront could be made esthetically attractive. The levee could be widened, and paved as a scenic roadway along the riverfront. Another possibility would be to convert the levee to a bicycle path throughout the riverfront area. The necessity for duplication of existing structures would be eliminated except for a few minor bypass facilities.

Reliability. This alternative would be less difficult to operate than other plans. The flow of liquid into the reservoir would be by gravity and would not depend on the operation of mechanical devices such as gates. Pumps and aerators would be controlled automatically with devices that sense the levels in the reservoir. Stand by generating facilities for providing power to the pumps and aerators would further improve the reliability of all the alternatives refined in the chapter.

Flexibility. This alternative can be implemented with the reservoirs providing limited protection at low costs while maintaining the capability to increase the extent of protection at any time. Additional protection in the future would be provided by increasing levee heights for increased capacity to contain storms of a larger recurrence interval. Advanced levels of treatment would be more readily obtainable, since all of the stored overflows would be pumped through the existing interceptor system for treatment at a central plant.

Institutional and Legal Constraints. The system would be contained within the City of Omaha geographical boundaries. It would be operated and controlled by city personnel. No infringement of land outside the sewerage system would be required, except for small portions in the

Carter Lake area. Some properties would have to be purchased by an authorized institution or agency for use as storage reservoirs. Potential problems may be encountered with the Riverfront Development Program, which would conflict with the land use requirements for this alternative plan.

Time of Construction. The construction time required for this alternative plan would be the shortest of the plans studied. The dikes could be constructed within an estimated 2-year period. Modifications to the existing diversion structures and the addition of pumping stations and conveyance conduits would take place simultaneously with the construction of the reservoirs. Treatment facilities at the Missouri River treatment plant would also be constructed simultaneously.

Implementation. A projected implementation schedule for Alternative 2 is shown on Plate 30. It is estimated that this alternative plan could become operational by June 1, 1979 provided the selection of this plan for implementation is made no later than Oct. 1, 1975.

Environmental Assessment

Implementation of Alternative 2 will have some economic effects on the Omaha area. With respect to changes in output and production, the pre-storage conveyance

facilities and the treatment facilities will require concrete and steel resources. Some earth spoil will be generated during construction of the reservoir and rock and riprap will be needed for wave protection on the reservoir sides. Public contract construction will increase during the construction period. The costs of implementation will be shared by the community and the federal government, with the local community paying $4.7 million annually for operation, maintenance, and replacement. Local employment would be increased during construction.

With respect to the natural environment, the implementation of Alternative 2 will improve the Missouri River water quality by reducing the intermittent loadings of fecal coliforms, BOD_5, suspended solids, nutrients, toxic substances, dissolved solids, metals, and chemicals. Air quality may be adversely affected by odor problems that could result if the aerators fail to maintain aerobic conditions in the reservoirs. Land use may be affected by sludge disposal, disposal of excess excavated materials, and requirements for storage and service of equipment during construction. This alternative will not alter the natural habitat of the plant, fish, and wildlife in the area. The proposed reservoir sites are neither designated forest preserves or wildlife sancturaries. The surface reservoirs along the river, south of Burt-Izard would be overlooked by

residents in the Riverfront New-town-in-town which could result in potential aesthetic problems.

The lack of public acceptance for aerated lagoons in or near urban areas could be a potential problem. Careful operation and maintenance of the system would help to overcome many of the objections. The lagoons could be designed with consideration given to landscaping and adjacent green areas, including recreational parks and playgrounds. Unsightly debris and floating material in the incoming wastewater could be separated initially in an enclosed structure. A comprehensive educational program aimed at public involvement would be required of any alternative using aerated reservoirs and lagoons in and near urban areas.

Alternative 4A - Deep Tunnel North to Ground Level Storage

This alternative plan utilizes a deep tunnel system to convey combined sewer overflows to a ground level reservoir located north of Council Bluffs in Iowa. Combined sewer overflows would be diverted to vertical drop shafts and conveyed to a tunnel located approximately 500 feet below the ground surface. This tunnel would be bored through a sound rock formation, paralleling the existing interceptor. The flow would be from the Monroe Street outfall north to the Minne Lusa outfall, then underneath the Missouri River to the reservoir in Iowa. The difference in elevation between the land along the western edge of the Missouri River and the low-lying area north of Council Bluffs would be sufficient to allow the water to flow by gravity to a ground level storage reservoir. Captured overflow would then be pumped back across the Missouri River into the interceptor and conveyed to the Missouri River plant for treatment.

The basic components of this system are the drop shafts, deep tunnel, pumping stations, ground level storage reservoir, and treatment facilities.

Drop Shafts

Combined sewer overflow would be intercepted near the outfalls at the Missouri River, as shown on plate 33. Diversion and control structures at these points would

direct overflows into vertical drop shafts for delivery to the tunnel system. Each drop shaft structure would consist of an entrance chamber, vent chamber, drop shaft, dividing wall, air vent, and air separation chamber. The entrance chamber would house the diversion and control structures. The drop shaft would be separated by a dividing wall to act as an air vent. Entrained air would be removed in the air separation chamber at the bottom of the drop shaft and returned to the surface through the air vent. A typical drop shaft structure is shown on Plate 34.

Cost estimates were based on a total of 20 vertical drop shafts, ranging from 8 to 18 feet in diameter for the 5-year recurrence interval.

Conveyance Tunnel

A circular unlined tunnel, bored into the Mississippian formation, approximately 500 feet below the ground surface, would convey the combined sewer overflow from the outfall points to the reservoir. This tunnel would range in diameter from 15 to 40 feet for the 5-year recurrence interval. From the Minne Lusa outlet, the tunnel would extend east for about 5,000 feet to an exit shaft surfacing at a ground level reservoir on the east side of the Missouri River.

The circular tunnel would be unlined and sloped to provide self-cleansing velocities during overflow events less than the design event. Some grouting and rock-bolting

may be necessary to minimize infiltration to the tunnel and to maintain the structural soundness of the tunnel system.

Velocities in the tunnel would be less than 15 feet per second but some loosening of material from the tunnel wall can still be expected. To capture rocks and any other large debris which may enter the tunnel system, rock traps, similar to the one shown on Plate 35, would be located midway and at the end of the tunnel system.

It was assumed that an unlined tunnel could be bored in the Mississippian formation by a tunneling machine (mole). The small amount of deep boring data available indicates that this formation is similar to a massive limestone formation in the Chicago area where such construction was carried out successfully. Concepts, cost estimates, and construction time are based on Chicago designs and experience.

A tunnel located in shallower rock formations would need to be lined for structural support and to eliminate the possibility of ground-water infiltration or exfiltration of combined sewage into the aquifers. The deep tunnel would be located below any useable aquifers and not present a pollution hazard.

Lift Station

A lift station at the exit shaft would be required to dewater the tunnel within 3 to 4 days after the storm.

Liquid would be pumped out of the tunnel and into the ground level storage reservoir at a rate of approximately 60 mgd. The lift station would include facilities such as trash racks, pumps, discharge piping, valves, and other items as shown on Plate 36. An elevator shaft would be installed to provide access to the pumping facilities.

No grit removal facilities would be associated with the lift station. The grit would be pumped to the ground level storage. This may shorten the lift of the pumps, but would eliminate the problem of operating a grit removal facility 500 feet below the ground surface.

<u>Ground Level Surface Storage Reservoir</u>

The ground level reservior would provide storage for the collected overflows. For the 5-year recurrence interval, a total of 3660 A-ft of storage would be required. Since the tunnel system would contain approximately 730 A-ft of storage, 2930 A-ft would be required in the ground level reservoir. After the storm or runoff event has subsided, a check gate at the inlet to the reservoir would prevent flow of liquid back into the tunnel system. As the reservoir is pumped out, the tunnel system would be emptied into the reservoir. Thus, the liquid level in the reservoir would remain unchanged for approximately 3 to 4 days as the tunnel is being dewatered.

The ground level reservoir could also be constructed as a multi-cell storage facility. The tunnel and first cell would be constructed to store overflow from the 1-year recurrence interval event. Additional cells would be empty except when the 1-year event was exceeded. This would reduce the problems associated with grit and sludge removal in a larger reservoir and reduce the number of aerators needed to maintain aerobic conditions.

The liquid level elevation in the ground surface reservoir would be approximately 980 ft msl. It is anticipated that the maximum liquid depth in this reservoir would be approximately 15 ft. Thus, for larger design recurrence intervals, the area would be increased proportionately to obtain the required storage volume. For the 5-year recurrence interval, the area required for the reservoir would be about 200 acres. The surface reservoir would have a low profile with grass-covered side slopes. Typical dimensions and an artist's conceptual layout of the reservoir are shown on Plate 37.

<u>Treatment Facilities</u>

Treatment of the stored combined sewer overflow could be accomplished at the reservoir site with an aerated lagoon, or the stored overflow could be pumped back through the interceptor to the Missouri River treatment plant. The cost of a contact-stabilization system at the Missouri River

or lowered to increase or decrease the flow to the interceptor system. During overflow events greater than the design recurrence interval, relief gates near the diversion structures would allow the excess to overflow to the Missouri River when the volume in the reservoir and tunnel reaches the maximum allowable level. This would prevent excessive surcharging and local flooding.

Periodic maintenance would be required to remove the rocks, gravel and accumulated grit near the diversion structures and rock traps. Trash racks at the diversion structures would be manually cleaned while hydraulic gates would have to be routinely checked because of infrequent use. Drop shafts and air vents must be periodically cleaned to remove debris and scum buildup. The tunnel system would also require periodic maintenance because of erosion and slime growth.

Sludge and grit would accumulate in the storage reservoir and would be cleaned out annually each November. Hydraulic dredges would be used, as the liquid level is lowered, to remove the settled sludge and grit. The removed material would then be barged to the existing treatment plant for disposal by incineration. After flushing, the reservoir would be drained with the aerators in place, resting on concrete pads.

treatment plant would be less than an aerated lagoon adjacent to the reservoir site.

System Performance and Operation

At the beginning of an overflow event, overflow would be directed to the tunnel through drop shafts. Debris that may have been carried into the combined sewer system would be prevented from entering the drop shafts by trash racks. After the tunnel and drop shafts had filled, overflow from the exit shaft would begin to fill the ground level surface reservoir. When the reservoir was filled to capacity, the maximum water elevation in the drop shafts and trunk sewers would be attained. This is illustrated by the intersection of the hydraulic grade line with the existing topography of the study area as shown on Plate 33. Any area between this line of intersection and the interceptor would have to have sewer separation and rehabilitation to prevent local flooding problems druing the period of maximum surcharge. Sewer separation for less than 20 acres is included in the total costs shown for this alternative plan.

Diversion structures would operate when the combined sewer flow exceeds the capacity of the interceptor system. This would occur for approximately 75 percent of the rainfall events during the spring, summer, and fall months. An overflow weir in the diversion structure could be designed with sufficient flexibility so that the weir could be raised

Special Considerations

The use of in-system attenuation, sewer separation, and upstream retention as elements of this alternative plan was also considered. Individual sewer service areas were analyzed to determine the advantages and disadvantages of these elements. The use of sewer separation or in-system attenuation appears uneconomical since reduction in the overall cost of conveyance, storage, and treatment facilities would not offset the additional expense. The use of upstream retention offers the greatest potential. The cost of the tunnel between the drop shaft at the Grace Street overflow point and the drop shaft at the Minne Lusa overflow point might be substantially reduced with the use of upstream retension through the reduction of peak flow rates in this section of the deep tunnel. The cost of this alternative plan could be reduced if upstream retention sites using storage that costs less than $30,000 per acre-foot could be found in the Grace Street Service area. The final location of upstream retention sites in the Grace Street service area would depend upon local flooding problems, availability of land, and the possible urban renewal projects that may take place under the Riverfront Development Program.

System Cost and Economic Considerations

The estimated cost and economic evaluation of this alternative plan were based upon the location of the drop

shafts, deep tunnel, and surface storage reservoir as shown on Plate 33. The total construction cost for the major components is estimated to be $144.6 million. Annual operation and maintenance costs are estimated to be $5.1 million.

If this system were designed to store and treat sufficient volume for the 1-year recurrence interval, then the construction cost could be decreased by approximately $48 million, to a total of $96.5 million. Comparable annual costs would decrease insignificantly.

Construction costs described above include necessary treatment to meet Level 1 or secondary treatment. However, if a higher degree of treatment is desired, then incremental costs must be added to achieve these levels. It would require an additional $30 million to achieve Level 2 treatment, utilizing a tertiary treatment and nutrient removal facilities. Additional advanced waste treatment to meet background water quality objectives (Level 3) would require additional processes, estimated at a cost of $30 million over the cost of Level 2 treatment.

Associated Effects

This alternative plan requires consideration of several associated effects as additional factors in the plan evaluation. These associated effects and considerations are discussed in the following section.

failures in the existing interceptor system. In the event of such a mechanical failure, the dry-weather flow could be diverted to a drop shaft and tunnel. This additional load on the ground level reservoir would slightly reduce the volume available to control the design event.

Reliability. The operation of this system does not depend upon pumping the peak overflow rates. The major mechanical facilities subject to breakdown are the pumping stations that pump stored overflows to treatment or dewater the tunnel. However, the tunnel is large enough to store almost one half of the overflow events that occur during the year. Thus, there would be capacity in the tunnel for short term sotrage if mechanical problems occur with the pumping station used to empty the surface reservoir.

Flexibility. This alternative could easily be expanded or modified to handle events of larger recurrence intervals by increasing the volume of the storage reservoir and by increasing the treatment capabilities at the treatment plant. Although the conveyance tunnel could not easily be enlarged to handle higher peak flows for larger recurrence intervals, the addition of upstream retention to lower these peak flows could be used for that purpose.

A constraint on this system is the capacity of the existing interceptor. If the system were sized to handle larger recurrence interval events, the pump-out rate from

Energy. The major components utilizing electrical power as a source of energy for this alternative plan would be the aeration equipment in the reservoir, the pumping station used to dewater the deep tunnel and the reservoir, and the bio-treat system. The power cost is estimated to be $2.0 million per year.

Manpower. This alternative plan would ge almost fully automated. The estimated number of personnel would be 23 full-time maintenance and operation employees. During the winter season, there may be slight increases in personnel required for servicing the tunnel system. Operation of these systems could, however, continue during labor strikes and shutdowns.

Resources. This alternative plan could be effective in conserving existing resources. The only land required is the surface storage reservoir in an agricultural area. The conveyance tunnel to that storage facility would be located underground. The pump-back facilities would primarily consist of the existing interceptor system. The force main from the storage reservoir to the existing interceptor system would be located underground. The mined rock obtained from the construction of the tunnel would offer potential resource that might be sold as a construction material.

Multi-use Potential. The system could be used to intercept dry-weather overflows which occur due to mechanical

the storage reservoir would be increased or the detention time in the reservoir would have to be increased. If the pump-out rate were increased, capacity of the existing interceptor system which is now available for future industrial flow would be reduced as shown in Table VII-3.

Institutional and legal constraints. The drop shafts and tunnel system would be located below ground on the Nebraska side of the Missouri River within the city limits of Omaha. However, the ground level surface reservoir would be located in Iowa. Thus, some institutional and legal problems may arise if land in Iowa is to be used for alleviating pollution problems in Omaha, Nebraska.

Time of Construction. The system could be constructed in phases, with the drop shafts and diversion structures to be built initially. This could be followed by construction of the deep tunnel and surface storage reservoir. The deep tunnel would require a period of approximately two years to complete. However, if the tunnel were constructed by starting at more than one location and if three shifts per day were utilized, the tunnel could be constructed in about one year. The treatment facilities could be constructed at the same time with the tunnel and reservoir facilities. These treatment facilities could also be constructed at the same time with the upgraded Missouri River treatment plant. Construction of the treatment facilities may take from 12 to 18 months. Drop shafts and diversion structures could

Table VII-3

Alternate 4A: CONSTRAINTS ON FUTURE INDUSTRIAL FLOW

Interceptor Location	Sewer Capacity (MGD)	Projected 2020 flow (MGD) 1/	Capacity For Combined & Future Industrial Flow	Pump-out Rate from Storage Reservoir 1-yr Event	2-yr Event	5-yr Event	10-yr Event	Available Capacity in Interceptor 1-yr Event	2-yr Event	5-yr Event	10-yr Event
Minne Lusa to Grace	56.3	6.2	50.1	37	46.5	61	71	13.1	3.6	(-10.9)	(-20.9)
Grace To Burt Izard	42.5	10.7	31.8	37	46.5	61	71	(-5.2)	(-14.7)	(-29.2)	(-39.2)
Burt Izard to Leavenworth	52.3	6.2	36.1	37	46.5	61	71	(-0.9)	(-10.4)	(-24.9)	(-34.9)
Leavenworth to Pierce	95.4	26.4	69.0	37	46.5	61	71	32.0	22.5	8.0	(- 2.0)
Pierce to Hickory	97.5	26.88*	70.6	37	46.5	61	71	33.6	24.1	9.6	(- 0.4)
Hickory to Martha	102.7	27.32*	75.4	37	46.5	61	71	38.4	28.9	14.4	4.4
Martha to Grover	102.7	27.76*	74.9	37	46.5	61	71	37.9	28.4	13.9	3.9
Grover to Riverview	102.8	28.20*	74.6	37	46.5	61	71	37.6	28.1	31.6	3.6
Riverview to Homer	106.3	28.64*	77.7	37	46.5	61	71	40.7	31.2	16.7	6.7
Homer to Missouri	106.3	29.08*	77.2	37	46.5	61	71	40.2	30.7	16.2	6.2
Missouri to STP	108.0	29.50	78.5	37	46.5	61	71	41.5	32.0	17.5	7.5

1/ Consists of Domestic and Commercial Flow

* Linerally Interpolated

With respect to the natural environment, the implementation of Alternative 4A will improve the Missouri River water quality by reducing the intermittent loadings of fecal coliforms, BOD_5, suspended solids, nutrients, toxic substances, dissolved soilds, metals, and chemicals. Air quality may be adversely affected by dust and particulates during construction and odor problems may occur if there are mechanical failures of aerators in the storage reservoirs. Land uses may be affected by slude disposal, disposal of excess excavated or mined materials, storage of construction aggregate, and storage and servicing of construction equipment.

Alternative 4B - Excavated Storage North - Deep Tunnel to Ground Level Storage

The Omaha Missouri River sewerage system can be considered as two distinct zones, with different topographic features. The topography in the Mormon Street, Minne Lusa Street, and Grace Street sewer service areas does not include steep bluffs along the interceptor such as occur along the South Omaha interceptor. Thus, ground level in these northern service areas rises gradually compared to the sharp increase in grade near the river south of the downtown Omaha area. Based on this topographic difference, this plan would use two excavated, open reservoirs in the Northern zone to store the overflow from the Mormon, Minne Lusa, and Grace Street service areas. In the southern zone, the

also be constructed within 12 to 18 months. The total construction time for the complete system is estimated at approximately 30 months.

Implementation. A projected implementation schedule for Alternative 4A is shown on Plate 30. It is estimated that this alternative plan would become operational by April 1, 1980 provided the selection of this plan is made no later then October 1, 1975.

Environmental Assessment

Implementation of Alternative 4A will have some economic effects on the Omaha area. With respect to changes in production and output, the construction of the deep tunnel system will require concrete and steel, but will produce earth and rock spoil. The rock may be marketed as a concrete aggregate or as crushed stone, and the earth used as landfill. Public contract construction will increase during the construction period. The costs of implementation would be shared by the community and the federal government, with the community paying more than $4 million annually for operation, maintenance, and replacement. Some local employment would be increased during construction, but much of the construction below ground would be by contractors from outside the local area. Approximately 200 acres of agricultural land will be taken out of production to construct the reservoir.

elevation differential between the bluffs and the river would provide energy to convey overflows from the Burt-Izard Street service area south to a ground level reservoir in Iowa. Conveyance for the southern zone would be by deep tunnel, constructed several hundred feet below grade in sound rock formations. In this manner, all of the southern zone overflows would be conveyed to the reservoir by gravity flow as in alternative 4A.

The major components required would be pre-storage conveyance, storage facilities, post-storage conveyance systems, and treatment facilities. Of these major components, only the treatment facilities are common to both the north and south zones. The location of these components are shown on Plate 38.

Pre-storage Conveyance Systems

Conveyance of overflow to storage facilities requires that the conveyance systems be sized to convey peak flow rates that occur during an overflow event. Because these peak overflow rates are on the order of 100 times the dry-weather flow rates, the pre-storage conveyance systems require special consideration.

Northern Zone. In the northern zone, the relatively short distances between the overflow points and the excavated storage reservoir allow for use of large box sewers or open channel conveyance systems. Overflows from the Mormon

Street area would be conveyed by a box sewer to the Minne Lusa Street outfall. From the Minne Lusa Street outfall, a much larger box sewer, approximately 2500 feet long, would convey overflows from both service areas to the storage reservoir. For the 5-year recurrence interval, this box sewer is estimated to be 18 ft by 35 ft. The use of an open channel to convey the overflow to the storage reservoir would be about one-third the cost of the box sewer, but the channel would traverse through a residential area and would not be acceptable.

Overflows from the Grace Street service area would be conveyed in the existing Grace Street outfall channel and directed into the storage reservoir by means of a diversion structure.

Southern Zone. Overflows from the service areas south of the Grace Street area would be diverted into vertical drop shafts and a deep tunnel system. Each drop shaft structure would consist of an entrance chamber, drop shaft with a dividing wall and air vent, and air separation chamber. The details of the system would be similar to Alternative 4A described previously.

The drop shaft diameters for the 5-year recurrence interval would vary from 5 to 18 ft in diameter. The 11 shafts would be circular and concrete lines. Depths of the drop shafts would average about 500 ft. depending upon the best rock formation for construction of the tunnel system.

capacity to store the design event and would be used infrequently so aeration equipment could be kept on stand-by to operate only for the larger storm events.

Northern Zone. The northern excavated reservoir would store overflows from the Mormon and Minne Lusa Street service areas. This reservoir would cover a surface area of about 74 acres to contain the 5-year event as shown on Plate 38. The entire storage volume would be below the existing grade elevation to allow gravity flow into the reservoir. The design water surface elevation would be about 970 ft, mean sea level (msl), and the reservoir bottom would be about 955 ft, msl. The reservoir would have a clay lining, underdrain system, and riprap to protect against wave action on the inside side slopes as shown on Plate 39. The lining and underdrain system are necessary to prevent uplift because of the high ground water conditions in the area.

The overflow from the Grace Street service area would be stored in a reservoir similar to the Minne Lusa reservoir. The location of the reservoir, as shown on Plate 38, would be between the oil storage tanks and Abbott Drive. For the 5-year event the surface area covered by this reservoir would be about 26 acres. The design water surface elevation of the reservoir would be about 990 ft, msl, and the maximum operating liquid depth would be 15 ft.

The tunnel system would extend south from the Burt-Izard drop shaft to the South Omaha drop shaft and would parallel the South Omaha interceptor. Overflow from the Monroe Street and "U" Street service areas would flow north through a smaller tunnel joining the larger tunnel below the South Omaha outfall. From this point the tunnel would extend east under the Missouri River for about 6000 ft. At the end of the deep tunnel there would be an exit shaft to the surface storage reservoir. The tunnel diameter would vary from 18 to 30 ft.

After an overflow event, the tunnel system would be dewatered, usually within 3 to 4 days, to provide storage capacity for the next event. A lift station at the exit shaft would include facilities such as trash racks, pumps, discharge piping, valves, and other items, as shown on Plate 36.

Storage Facilities

Storage reservoirs for both zones will be located at the ground surface. These reservoirs would be aerated with surface aerators to maintain settleable solids in suspension and to maintain aerobic conditions. The reservoirs could be multi-celled to minimize the number of surface aerators operating during periods when overflow events normally occur. The first cell could be sized to contain the estimated volume of overflow for the event with a 1-year recurrence interval. The additional cells would provide reserve

Southern Zone. The single reservoir for the storage of overflows from the southern zone would be located directly east of the South Omaha outfall, on the Iowa side of the Missouri River, as shown on Plate 38. The bottom of this clay-lined reservoir would be at about elevation 970 ft, msl, the existing ground surface elevation. For the 5-year event the surface area requirement for this reservoir would be 144 acres, based upon a design water elevation of 985 ft, msl. The details of this reservoir would be similar to the ground level reservoir described in the previous alternative Plan 4A.

Post-Storage Conveyance Facilities

A pumping station would be provided at each reservoir. The Minne Lusa-Mormon Street pumping station would discharge the stored overflow to the existing North Omaha interceptor. The stored overflow in the Grace Street reservoir would be pumped to the South Omaha interceptor. Overflows stored in the ground level reservoir in Iowa would be pumped out through a force main across the Missouri River to the South Omaha interceptor.

Treatment Facilities

All stored overflow would be treated at the upgraded Missouri River treatment plant. For planning purposes, the equivalent of secondary treatment would be provided by a contact stabilization system. Higher levels of treatment

could be obtained by constructing additional unit processes such as ammonia stripping towers, multi-media filtration, alum addition for phosphorus removal, ion exchange, and/or activated carbon.

System Operation and Performance

Combined sewer overflow would be diverted to the pre-storage conveyance facilities whenever the interceptor system was overloaded. The storage reservoirs could be constructed as a multi-cell facility. Then the overflow would enter the first cells where grit and settleable solids would be deposited. Liquid levels in the first cells would vary with the amount of overflow captured. After each overflow event the stored overflow would be pumped out, leaving a liquid cover of several feet to help minimize odors. The other cells in reservoirs would receive flow less frequently thus requiring less maintenance. Hydraulic dredges would be used to remove the accumulated grit and sludge each November. The dredged material would then be barged from each reservoir to the existing Missouri River treatment plant for incineration. After flushing the reservoirs, the water can be drained and the aerators left in place, resting on concrete pads. In this manner, freezing problems and odor problems would be minimized.

The pre-storage conveyance facilities to the northern reservoirs would be pumped out by the same pumping station that empties the storage reservoirs.

Leavenworth, South Omaha, and Monroe Street service areas provided the cost if less than $30,000 per A-ft of storage volume.

System Cost and Economic Considerations

For this alternative plan, the location of the major components is shown in Plate 38. The estimated construction cost for construction for the 5-year recurrence interval of the major components, as shown on Plate 27, is estimated to be $111.0 million. The annual operation and maintenance costs for this alternative plan are estimated to be $5.1 million.

If this system were designed to provide protection from the 1-year recurrence event, the total construction cost would be decreased by approximately $31 million to a cost of $80.2 million. Comparable annual costs would decrease insignificantly.

The construction costs described above include the treatment required to meet Level 1 or secondary treatment. However, if a higher degree of treatment is desired, then an incremental cost would be required similar to the alternatives previously described.

Associated Effects

The evaluation of alternative plans requires consideration of several associated effects as a means of plan com-

The southern zone components would operate the same as the tunnel and reservoir system as in Alternative 4A described previously.

Special Considerations

The use of sewer separation, upstream storage and in-system attenuation were considered as possible elements of this alternative plan. In-system attenuation offers no economic savings because of its relatively high cost when compared with the cost of the basic plan components.

Sewer separation in the Burt-Izard service area is, however, an attractive element for this alternative plan. Separation in this area is based on the intersection of the hydraulic grade line for the deep tunnel system and the topographic contours, as shown on Plate 38, indicating that part of the Burt-Izard service area would have local flooding problems during the design event. The area required for separation would be approximately 260

Upstream retention may also be a possible element to reduce peak flows in the tunnel system and the required storage volume of the storage reservoir in Iowa. Whenever local flooding problems in the southern zone could be alleviated by an upstream retention basin, use of such a basin would reduce the overall cost of this alternative plan. Cost reduction potential for the total plan would be greatest for upstream retention sites in the Burt-Izard,

parison. These considerations are discussed below for this alternative plan.

Energy. The major components using electrical power as an energy source are aeration equipment, pumping stations, and bio-treat system. The power cost is estimated to be $1.9 million per year.

Manpower. This alternative plan could be almost entirely automated. The total number of operating and maintenance personnel is estimated at 23 full time employees. During the winter, a slight increase in maintenance and cleanup personnel would be required to service the tunnel. Operation of these facilities could, however, continue during labor strikes and shutdowns.

Resources. The only land requirements for this plan are the storage reservoirs. The larger southern reservoir of 144 acres would be located on what is now agricultural land. The northern sites would preempt land planned for industrial use. Spoil from the excavated reservoirs in the North would have to be disposed of by landfill. The excavated rock from the tunnel could supplement the present local sources of aggregate and possibly be sold locally.

Multi-use Potential. This alternative could capture and treat not only combined sewer overflows, but also dry-weather sewage by-passes caused by mechanical failure in the present system. In the event of such a failure, dry-

weather flow could be diverted to storage and subsequent treatment.

<u>Reliability</u>. The operation of this plan is not dependent upon pumping the peak overflow rates. All flow to storage is conveyed by a gravity system. The major mechanical facilities subject to break-down are the pumping stations that pump the stored overflows to treatment or dewater the tunnel. Without operation of these pumps, the plan would still provide primary treatment of the overflow in the storage reservoirs. Power outages could result in odor productng conditions for short periods in the aerated storage reservoirs. The effect would be similar for all alternatives since all plans require storage supplemented by aeration.

<u>Flexibility.</u> This alternative could easily be expanded or modified to handle events with larger recurence intervals by increasing the volume of the storage reservoirs and the treatment capacity. The capacity of conveyance facilities could not be as easily increased, but upstream retention could be used for this purpose. In addition, higher levels of treatment could be provided at the treatment plant with a degree of flexibility similar to the planned upgraded Missouri River treatment plant.

The capacity of the existing interceptor which is now available for future industrial flow would be reduced with

larger design recurrence intervals as shown in Table VII-4. This effect could be offset by increasing the storage volume and reducing the treatment rate.

Institutional and Legal Constraints. The southern zone storage reservoir in Iowa and the Grace Street reservoir will be outside the city limits of Omaha. Both of these reservoirs would be located in Iowa. Institutional and legal problems may arise if land in Iowa is used to alleviate pollution problems in Omaha.

Time of Construction. The system components for this plan could be constructed in several phases, or simultaneously. Construction in the northern zone can be preformed independently of the southern zone. A construction time of 12 to 18 months is estimated for the northern conveyance and storage facilities. A construction time of 12 to 18 months is also estimated for the treatment conveyance and storage facilities in the southern zone. The construction time, estimated for the tunnel system, is based upon working three shifts per day and constructing drop shafts independently of the tunnel. Thus, the entire system could be constructed in approximately 30 months.

Implementation. A projected implementation schedule for Alternative 4B is shown on Plate 30. It is estimated that this alternative plan would become operational by April 1, 1980 provided the selection of this plan is made

Table VII-4

ALTERNATIVE 4B: CONSTRAINTS ON FUTURE INDUSTRIAL FLOW

Interceptor Location	Sewer Capacity (MGD)	Projected 2020 Flow (MGD)1/	Capacity For Combined & Future Industrial Flow	Pump-out Rate from Storage Reservoir 1-yr Event	2-yr Event	5-yr Event	10-yr Event	Available Capacity in Interceptor 1-yr Event	2-yr Event	5-yr Event	10-yr Event
Minne Lusa to Grace	56.3	6.2	50.1	11	14	19	22	39.1	36.1	31.1	28.1
Grace to Burt Izard	42.5	10.7	31.8	15	19	25.5	29.6	16.8	12.8	6.3	2.2
Burt Izard to Leavenworth	52.3	16.2	36.1	15	19	25.5	29.6	21.1	17.1	10.6	6.5
Leavenworth to Pierce	95.4	26.4	69.0	15	19	25.5	29.6	54.0	50.0	43.5	39.4
Pierce to Hickory	97.5	26.88*	70.6	15	19	25.5	29.6	55.6	51.6	45.1	41.0
Hickory to Martha	102.7	27.32*	75.4	15	19	25.5	29.6	60.4	56.4	49.9	45.8
Martha to Grover	102.7	27.76*	74.9	15	19	25.5	29.6	59.9	55.9	49.4	45.3
Grover to Riverview	102.8	28.20*	74.6	15	19	25.5	29.6	59.6	55.6	49.1	45.0
Riverview to Homer	106.3	28.64*	77.7	15	19	25.5	29.6	62.7	58.7	52.2	48.1
Homer to Missouri	106.3	29.08*	77.2	15	19	25.5	29.6	62.2	58.2	51.7	47.6
Missouri to STP	108.0	29.50	78.5	15	19	25.5	29.6	63.5	59.5	53.0	48.9

1/ Consists of Domestic and Commercial Flow

* Linerally Interpolated

no later than October 1, 1975.

Environmental Assessment

Implementation of Alternative 4B will have minor economic effects on the Omaha area. With respect to changes in production and output, the construction of the tunnel system will require concrete and steel while producing earth and rock spoil. The rock may be marketed as a concrete aggregate or as crushed stone and the earth used as landfill. Some farmland will be taken out of production for the southern reservoir, and the reservoir in the northern zone will preempt industrial development. Public contract construction will increase during the construction period. Local employment will increase, but much of the construction work on the tunnel system would probably be by contractor from outside the Omaha area. The costs of implementation will be shared by the local community and the federal government with the local community paying more than $4 million annualy for operation, maintenance, and replacement.

With respect to the natural environment, the implementation of Alternative 4B would be similar to the impact discussed for the previous alternative plan.

Institutional and Legal Constraints. The southern zone storage reservoir in Iowa and the Grace Street reservoir will be outside the city limits of Omaha. Both of these reservoirs would be located in Iowa. Institutional and legal problems may arise if land in Iowa is used to alleviate pollution problems in Omaha.

Time of Construction. The system components for this plan could be constructed in several phases, or simultaneously. Construction in the northern zone can be performed independently of the southern zone. A construction time of 12 to 18 months is estimated for the northern conveyance and storage facilities. A construction time of 12 to 18 months is also estimated for the treatment conveyance and storage facilities in the southern zone. The construction time, estimated for the tunnel system, is based upon working three shifts per day and constructing drop shafts independently of the tunnel. Thus, the entire system could be constructed in approximately 30 months.

Implementation. A projected implementation schedule for Alternative 4B is shown on Plate 30. It is estimated that this alternative plan would become operational by April 1, 1980 provided the selection of this plan is made no later than October 1, 1975.

Environmental Assessment

Implementation of Alternative 4B will have minor economic effects on the Omaha area. With respect to changes in production and output, the construction of the tunnel system will require concrete and steel while producing earth and rock spoil. The rock may be marketed as a concrete aggregate or as crushed stone and the earth used as landfill. Some farmland will be taken out of production for the southern reservoir, and the reservoir in the northern zone will preempt industrial development. Public contract construction will increase during the construction period. Local employment will increase, but much of the construction work on the tunnel system would probably be by contractorfrom outside the Omaha area. The costs of implementation will be shared by the local community and the federal government with the local community paying more than $4 million annually for operation, maintenance, and replacement.

With respect to the natural environment, the implementation of Alternative 4B would be similar to the impact discussed for the previous alternative plan.

Alternative 5A - Deep Tunnel With Mined Storage

The basic concept of the deep tunnel and mined storage alternative was described earlier in the discussion of alternative concepts. Although this alternative was not one of the least cost concepts, it was decided to proceed with further evaluation in order to provide an alternate plan without ground level storage.

This alternative consists of a deep tunnel system for conveying combined sewer overflows to a storage reservoir mined in sound rock beneath the Missouri River treatment plant. The combined sewer overflow would be dropped through vertical shafts to an unlined tunnel that would generally parallel the Missouri River. The tunnel would be designed to flow full and utilize the available head to convey the overflows to a central mined storage reservoir about 500 feet below the surface. The mined reservoir would consist of large unlined chambers in the Mississippian geologic formation. The reservoir would be divided into two separate sections with the first used for solids separation and settling and the second as the main area for aerated storage.

The solids settling section would be sized to contain overflows from smaller storms, while retaining the majority of the solids generated by most storms. The excess from larger storms would overflow into the main storage reservoir. This partially biologically treated overflow would then be

pumped to the treatment facility at the ground surface at a constant rate.

Drop Shafts

The existing combined sewers would be intercepted near the outfalls at the Missouri River, as shown on Plate 40. Diversion or control structures at these points would divert overflow exceeding the existing interceptor capacity into the vertical drop shafts for delivery to the tunnel system as described for preceding alternatives. These shafts would differ from those in preceding alternatives in that they would be gated to prevent flooding of the mined reservoir when the reservoir is full. A total of 20 vertical shafts would require varying from 8 feet to 18 feet in diameter for the 5 year recurrence interval. Since the storage reservoir would be far below the ground surface, there would be no surcharging of sewers as in other deep tunnel alternatives.

Conveyance Tunnels

A circular unlined tunnel, similar in concept to preceding alternatives, ranging in diameter from 19 to 28 feet, and about ten miles long, would convey the flow from the vertical drop shafts to the mined storage facilities as shown on Plate 40. The tunnel would be designed for flow velocities up to 40 feet per second. The tunnel would also be designed with sufficient slope to maintain self cleansing velocities for the minor storms when the system would only

be partially full.

Mined Storage Reservoir

The mined storage reservoir would provide storage capacity for the overflows prior to treatment at the Missouri River plant. The reservoir would be constructed by high production mining methods. The room and pillar arrangement shown would make available a large number of working faces efficiently utilizing equipment and labor throughout the construction cycle. The galleries or chamber headings would be spaced approximately 200 feet apart and would be approximately 35 feet wide by 70 feet high consisting of approximately 68,000 lineal feet of galleries constructed in an honeycomb fasion. These dimensions would be dependent on the geologic characteristics of the rock. In the Mississippian formation, rock bolting may be adequate for structural integrity and lining of the reservoir should not be required.

The reservoir would be divided by overflow structures into two sections, a smaller section for settling and removal of solid materials and a larger section for aeration and detention of the remaining overflows. The settling chamber would receive flow directly from the conveyance tunnel, and would provide sufficient detention time to allow for deposition of grit and larger settlable materials. The process used would be similar to the primary treatment of raw sewage now used at the Missouri River treatment plant. The floor of the settling chamber would be concrete lined to accommodate mechanical equipment for sludge handling

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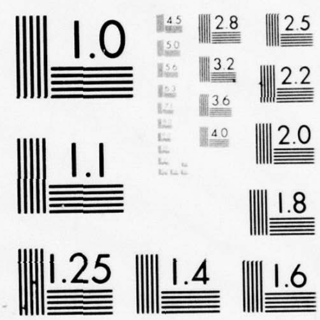

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and removal. High head pumps at one end of the chamber would be used to elevate the sludge to the treatment facility at the surface.

For large overflow events, the partially treated combined sewer overflows would spill from the settling chamber into the main storage area. The overflows would be held in storage and aerated until the settling chamber was emptied. The aerated main reservoir would then be pumped to the surface treatment facility. The storage reservoir volume for both sections would be approximately 3200 A-ft for the 5-year and about 1900 A-ft for the 1-year recurrence interval. The chambers would be ventilated and lighted to provide a safe atmosphere for inspection and maintenance.

Pumping Station

The pumping station for dewatering the reservoir would consist of approximately four pumps, sized to deliver a total of 60 mgd at maximum head. The station would include necessary facilities such as trash rack, suction piping, pumps, discharge piping, valves and other items, as shown in an illustration of a typical pump station on Plate 36. An elevator shaft would be installed to provide access to the pumping facility.

Treatment Facility

The wet-weather stored overflows would be treated by a standby process similar to contact stabilization, as in preceding alternatives. It is recognized, as indicated earlier,

that other treatment systems can also be used such as the activated sludge process, modified activated sludge, physical-chemical processes, and additional advanced waste treatment methods as required to meet the desired levels of treatment.

System Performance and Operation

The full storage and pumping capacity of the system would only be required at rare intervals. Very large events could overload the storage facility. Therefore, when the storage chambers are almost full, gates at the diversion structures would be required to prevent further inflow to the system. Under these circumstances the overflows would bypass to the Missouri River.

The diversion structures would operate when the flow in the sewers is greater than three to five times the dry-weather flow, depending on the sewer involved. This would occur during approximately 75% of the storms in the summer months. The overflow weir in the diversion structure would be designed so it could be raised or lowered to increase or decrease the flow to the treatment plant. Surcharging and local flooding could be prevented by allowing diversion to the drop shaft structures at less than overflow conditions.

Bar screens for removal of trash would be manually cleaned since high maintenance and service are required with mechanically cleaned screens. Maintenance would have to be performed at the structures to remove heavy rocks, gravel,

but again the savings in tunnel costs would not offset the costs for attenuation. The use of upstream retention basins could be used to reduce flooding in some areas and could prove to be attractive with further optimization of this alternative plan. Study of upstream retention basins in individual service areas show that use in the Minne-Lusa area, a significant contributor to the peak flow, should be considered further. The basins would decrease and attenuate the peak flow and thereby reduce the tunnel diameter to the mined storage facilities. Upstream retention should be considered if storage basins can be constructed for less than $30,000 per A-ft.

System Cost and Economic Considerations

The proposed layout of the conveyance tunnels, drop shafts, diversion structures, storage chambers, and treatment facilities is shown on Plate 40. The estimated costs for the major project components and facilities, are shown on Plate 27. The construction cost is estimated at $178.8 million for the 5-year recurrence interval. The total annual cost for operation and maintenance is $6.1 million.

If the system is designed to store and treat sufficient volume for the 1-year recurrence interval, then the cost would decrease to approximately $126.6 million or about $52 million less than the 5-year recurrence interval. The differential cost would prevent four additional overflow events in a five year period. This incremental protection represents approximately 65 tons of BOD discharged

and accumulated grit upstream of the diversion weirs. Drop shafts and air vents must be periodically cleaned to remove debris and scum buildup. Some tunnel maintenance would be required to prevent premature erosion or slime build-up.

Accumulated grit and solids in the settling chamber would be removed by mechanical means to sludge sumps and pumped to the surface for disposal at the Missouri River treatment plant. The overflows from the settling chamber would be aerated in the main storage chamber. Proper placement of aeration devices would provide for suspension of the lighter solid material. There would be very little accumulation of solids in the main reservoir. Manual flushing of the chambers might be needed occasionally. Although solids deposition in the main chamber would be minimal, some generation of noxious gases could develop. Therefore, a ventilation system would be provided for the storage chambers. Inspection and solids removal by manual means could be undertaken if necessary during the winter season when the facilities would be dry and clean.

Special Considerations

The use of in-system attenuation, sewer separation and upstream retention was also considered with this alternative plan. Individual service areas were analyzed to determine the individual merits of these elements. The derived benefits from sewer separation would be inadequate to offset the high cost involved in such a program. In-system attenuation would reduce the required tunnel diameter,

to the river per year. The corresponding reduction of pollutants for various recurrence intervals is summarized in Table VII-1. The table shows that the difference in impact on the Missouri River between the 1-year recurrence interval and the 5-year recurrence interval would not be substantial.

Costs described above include necessary treatment to meet level 1 or secondary treatment. It would require an additional $30 million dollars to achieve Level 2 treatment utilizing tertiary filtration and nutrient removal facilities. Additional advanced waste treatment to meet background water quality objectives would require processes at an estimated cost of $30 million.

Associated Effects

The evaluation of alternative plans require consideration of several associated effects to make plan comparison. These associated effects and considerations are discussed in the following section.

Energy. The major component utilizing electrical power as a source of energy for this alternative plan is the aeration and pumping equipment in the storage chambers. The total power cost for these units is estimated at $2.8 million per year.

Man-Power. The estimated number of operating personnel would be 19 full time maintenance and labor employees. During the dry winter season a slight increase in maintenance

and clean-up personnel would be utilized to service the tunnel and mined storage facility. The system should not be susceptible to labor strikes and shut-downs since a minimum number of operating personnel would be required to maintain the system even during wet weather periods.

Resources. This plan offers considerable potential for conservation of existing resources. Expensive land, highways and plans for the Riverfront Development Program would not be upset by this alternative since all of the major components for storage and conveyance would be located underground. There is also the possibility of integrating the construction of the mined storage reservoir with the production of rock and aggregate to meet current materials requirements in the Omaha area.

Multi-Use Potential. This alternative plan can be part of a phased construction program for the eventual implementation of a regional plan such as Alternative B as discussed next in this chapter. This alternative could also capture dry weather by-passes caused by mechanical failure in the existing interceptor system.

Reliability. The operation of the system is relatively simple, the services required are not excessive, and duplication of existing facilities is not needed. The minimum number of components required would help minimize service breakdowns. However, a backup diversion system would have to be provided at the drop shafts for by-pass of overflows

to the river if the diversion gates failed to close when the storage chamber is full.

Flexibility. This alternative plan could be staged to provide protection from events of larger recurrence intervals. The storage reservoir could be enlarged by mining more chambers and providing additional aeration facilities. The tunnels could be enlarged or upstream retention provided to reduce the peak flow rates.

Institutional and Legal Constraints. The entire system would be located within the geographical boundaries of the City of Omaha. The facilities would be operated and controlled by the city personnel. Political or geographical problems should not be encountered since the conveyance and storage facilities would be underground while the treatment facilities are located at the existing Missouri River Plant site.

Time of Construction. The system would be constructed in phases with the drop shaft and diversion structures built initially. This would be followed by the deep tunnel and mined storage facilities to be constructed simultaneously. With the use of tunneling mole machines the tunnel could be bored at a rate of 50 feet per day and completed in approximately two years. It is estimated that the mined storage facility would also be constructed during the same period. Treatment facilities, constructed simultaneously, would require 12 to 18 months. Drop shafts and diversion

structures would also be constructed in 12 to 18 months. Total construction time for the complete system is estimated at 2 to 5 years.

Implementation. A projected implementation schedule for Alternative 5A is shown on Plate 30. It is estimated that this alternative plan would become operational by October, 1981, provided the selection of this plan is made no later than October 1, 1975.

Environmental Assessment Considerations

Implementation of Alternative 5A will have some economic effects on the Omaha area. With respect to output and production, the construction of the deep tunnel system and mined storage facility will require concrete and steel, but it will also produce rock and earth spoil. The rock may be marketed as a concrete aggregate or as crushed stone and the earth used for landfill. Public contract construction will increase during the construction period. Some increase in local employment would be expected during construction, but it is expected that the construction of the tunnel system and mined storage reservoir would be undertaken by contractors outside the local area. The costs of implementation would be shared by the community and the federal government, with the local community paying more than $6 million annually for operation, maintenance and replacement.

With respect to the environment, this alternative plan has considerable advantages over the previous alternatives

since all the facilities would be underground. Therefore, there would be virtually no affect or impact on the surrounding area.

Alternative B - Deep Tunnel to Papillion Creek Sewage Treatment Plant

This plan is different in basic concept from other alternative plans in that combined sewer overflows would be conveyed to a new regional treatment plant. In almost all respects, this alternative plan, as shown on Plate 41, is the same as Alternative 5A except that the stored overflow would be conveyed from the mined storage facility through another deep tunnel to a point below the Papillion Creek sewage treatment plant. From this point the overflow would be pumped to the ground surface for treatment by conventional methods or land disposal. Drop shafts located at each overflow point, a deep tunnel, and a mined storage reservoir are identical to the previous.

This alternative plan, as presented, is incomplete because the treatment required would depend on the process selected for the Papillion Creek facility. Several considerations make this alternative attractive from the regional wastewater management point of view. Dry-weather primary effluent from the Missouri River plant could also be diverted through a drop shaft to the deep tunnel and conveyed to the Papillion Creek plant. This would eliminate

the need for the proposed Missouri River treatment plant addition. Also, by centralizing the wastewater, the possibility of disposal by land application is greatly enhanced.

The following discussion of this alternative plan includes only those facets of the alternative that differ from Alternative 5A. Costs, are presented for the alternative, exclusive of the treatment facilities.

Conveyance to Papillion Creek Plant

Overflow, stored in a mined storage reservoir beneath the Missouri River treatment plant, would be conveyed through a 6 ft diameter, unlined tunnel to a point 700 ft below the Papillion Creek treatment plant. From this point the flow would be lifted to the ground surface. The pumping facility required for this conveyance is a lift station at the end of the 6 ft diameter tunnel. Conveyance of the stored overflow, approximately 42,000 ft to the Papillion Creek treatment plant, would be by gravity.

Several alternative methods of conveying the overflow to the Papillion Creek site were considered. These methods included conveyance of the peak overflows directly to the Papillion Creek site in a deep tunnel and conveyance of the overflow in a force main near the ground surface. Attenuation of the flow rate in a mined storage reservoir at the end of the tunnel compared to installation of greater pumping capacity to handle the peak flow rates was also

the storage reservoir through the 6 ft diameter tunnel rather than pumped to the existing Missouri River treatment plant site. The addition of primary effluent to the tunnel would not alter the rate at which the reservoir is emptied.

Sludge and grit handling and disposal would be carried out in the same manner as described in Alternative 5A.

Special Considerations

The same discussion as given in Alternative 5A would also apply to this alternative plan.

System Cost and Economic Considerations

The total construction cost for this alternative plan (5-yr recurrence interval) is estimated to be \$189.5 million and the annual operation and maintenance cost is estimated to be \$5.8 million. These costs, as shown in Plate 27, include the cost for conveyance of the primary effluent along with stored overflows to the Papillion Creek site. Without the primary effluent, the construction cost would decrease by an estimated \$2.2 million, and the annual operation and maintenance cost, by another \$0.8 million. For the 1-year recurrence interval the construction would decrease to approximately \$139 million.

The cost of this alternative plan is greatly dependent upon the type of rock formations and depths that would be encountered during construction. The unit costs for tunnel

construction and mined storage that were used to estimate the cost of this alternative are representative for conditions encountered in the Chicago area.

Associated Effects

The evaluation of alternative plans requires consideration of several, associated affects as additional means of plan comparison. The associated affects are discussed below for this alternative.

Energy. The major facilities using electrical power for this alternative plan are the aeration facilities, the sludge pumps, and the pumping station under the Papillion Creek site. The annual electrical power cost is estimated to be $3.6 million. Stand-by power for primary effluent pumping only, utilizing either gas or oil, would be estimated at $0.6 million.

Man-power. This alternative would require a minimal operating staff and several full-time maintenance personnel. The estimated total number of personnel would be 12 employees, which doe not include the treatment plant requirements. During the winter season, a slight increase in maintenance personnel may be required for inspection and maintenance of the conveyance tunnels and the mined storage reservoir. Special attention may be directed toward the maintenance of the pumping station, which is the most critical component

of this alternative. The operation of this plan could, however, continue if a labor strike or shutdown were to occur.

Resources. All of the major components of this alternative would be located underground. None of the resources in the study area would be taxed by implementation of this alternative plan. The mined rock from construction of the tunnels and storage reservoir could be sold on the local market as an aggregate or disposed of by landfill. The treatment and disposal of the overflow could involve disposal on land which would provide irrigation water to farmland south and west of Omaha.

Multi-use potential. In addition to capturing combined sewer overflows, this alternative could also capture dry-weather by-passes caused by mechanical failure in the existing interceptor system. As previously mentioned, this alternative could also be easily modified to convey primary effluent from the Missouri River plant to the Papillion Creek site. This would further contribute to the potential use of spray irrigation for final effluent disposal.

Reliability. The major plan component that affects system reliability is the pumping station. Reliability of this pumping facility can be increased by the provision of duplicate facilities and special attention to its mainte-

nance. A backup diversion system at the drop shafts would also be required as in the previous Alternative 5A.

Flexibility. This alternative plan can be staged to provide protection from events of larger recurrence intervals. The storage reservoir can be enlarged by mining more chambers and providing additional aeration facilities. The tunnels could be duplicated or upstream retention could be provided to reduce the peak flow rates. The 6 ft diameter tunnel would not have to be duplicated unless it is required to handle extremely rare events.

Institutional and legal constraints. No serious legal and institutional constraints are expected to arise with the construction of the underground facilities. However, regionalization of waste treatment and dispoal on land may pose potential institutional and legal problems. These problems will be addressed in the regional waste-water management study.

Time of construction. The entire system, as described in this study, could be constructed in the same length of time as Alternative 5A, from 2 to 5 years. The drop shafts, tunnels, storage reservoir, and lift stations can be constructed almost independent of each other or in series, depending upon the desired time of construction.

Implementation. A projected implementation schedule for Alternative B is shown on Plate 30. It is estimated that this alternative plan would become operational by June 1, 1982, provided the selection of this plan is made no later than October 1, 1975.

Environmental Assessment Considerations

The environmental impact of this alternative plan on the surrounding area would be similar to the impact described for the previous plan, Alternative 5A.

Concluding Remarks

The objective of this study was to identify and formulate solutions consistent with PL 92-500 to the problems of pollution caused by combined sewer overflows in the Omaha-Missouri River sewerage system. The latter phase of this study consisted of refining selected alternative concepts to the level of alternative plans. All of the selected alternative plans would be fully capable of meeting the objectives of PL 92-500.

Comparison of Alternatives. Of the five selected alternative plans, diked storage along the levee, Alternative 2, would have the lowest construction cost, $71.1 million and lowest annual operation and maintenance cost, $4.7 million. The two plans utilizing a deep tunnel with ground

level storage in Iowa, Alternative 4A and 4B would be more expensive with construction costs of $145 and $111 million. Annual operation and maintenance costs would be $5.1 million for both alternatives. As an alternative to surface storage of any type, Deep Tunnel with Mined Storage, Alternative 5A, would be the most expensive with a construction cost of $179 million and annual operation and maintenance cost of $6.1 million. Alternative B, part of a regional concept, would have a construction cost of $190 million and an annual operation and maintenance cost of $5.6 million for the underground conveyance and storage facilities only.

Although Alternative 2 is the least cost plan, it probably has the most adverse aesthetic impact on the Omaha area. The alternative plans with deep tunnel and surface storage in Iowa, Alternatives 4A and 4B, would have considerably less visual impact on the area. In addition, these plans would have less conflict with the development and land use program along the riverfront. Alternative 5A with all storage deep underground would have virtually no adverse or environmental impacts.

All the alternative plans would provide the same levels of performance. For the same cost Alternative 2 could provide a considerably higher degree of treatment than the other plans. Alternative 2 also can be implemented sooner

and would require less time for construction than the other plans. This alternative could be constructed in stages more readily than the other alternatives. Based on the above considerations Alternative 2 would be the most suitable. However, if the adverse visual impact and preemptive use of land along the riverfront cannot be offset by the lower cost of Alternative 2 then a selection from one of the remaining plans would have to be made.

Some of the adverse visual impact could be reduced by covering the surface storage reservoirs. Air pressurized inflatable covers would not be practicle due to the large areas required for the reservoir. Rigid covers, such as fiberglass structural domes, have been used on trickling filters and could be adapted to the reservoirs. The installed cost is estimated at $0.5 million per surface acre. The total estimated cost for any surface storage alternative requiring 300 acres based on the 5-year recurrence interval would be approximately $150 million.

Those alternatives utilizing a deep tunnel would have an intangible benefit of potential marketability of the rock spoil. The dolemite limestone in the Mississippian geologic formation could be sold for concrete aggregate if the drill and blast method is used for constucting the tunnel. If a mole machine is used then the spoil would be of a chip like material and unsuitable as a

concrete aggregate. However, the material could still be sold as a clean select fill. The aggregate could be marketed for approximately $1.50 per cubic yard while the select fill would sell for approximatley $0.75 per cubic yard. The potential income for any of the alternatives using a deep tunnel would be appropiately one to two million dollars.

Several major conclusions derived in this study are discussed below.

1. The least cost treatment method is a modified type of contact-stabilization system intermittently operating as wet weather standby plant in conjunction with a continuously operating dry weather activated sludge plant.

2. Any plan with a 5-year design recurrence interval would completely control 99.6 percent of the overflow events and capture 98.9 percent of the flow. The BOD and suspended solids reduction would be approximately 68 and 75 percent based on providing secondary treatment.

3. The overflow discharge from a storage reservoir during an event exceeding the selected recurrence interval would average from 25 to 100 mg/l with an approximate average of 50 mg/l.

4. The incremental cost for providing the protection against storms greater than the 5-year event would be substantial. The 2-year or less recurrence interval would be the most cost effective selection.

5. Further study integrating water quality modeling of the Missouri River with optimization of the alternative plan components may demonstrate the selection below the 1-year event may be adequate for the Omaha Missouri River sewerage system.

6. The three elements considered for in-system attenuation, sewer separation, and upstream retention were either too costly for the benefits derived or provided only partial and insufficient solutions to the problem abatement from combined sewer overflows.

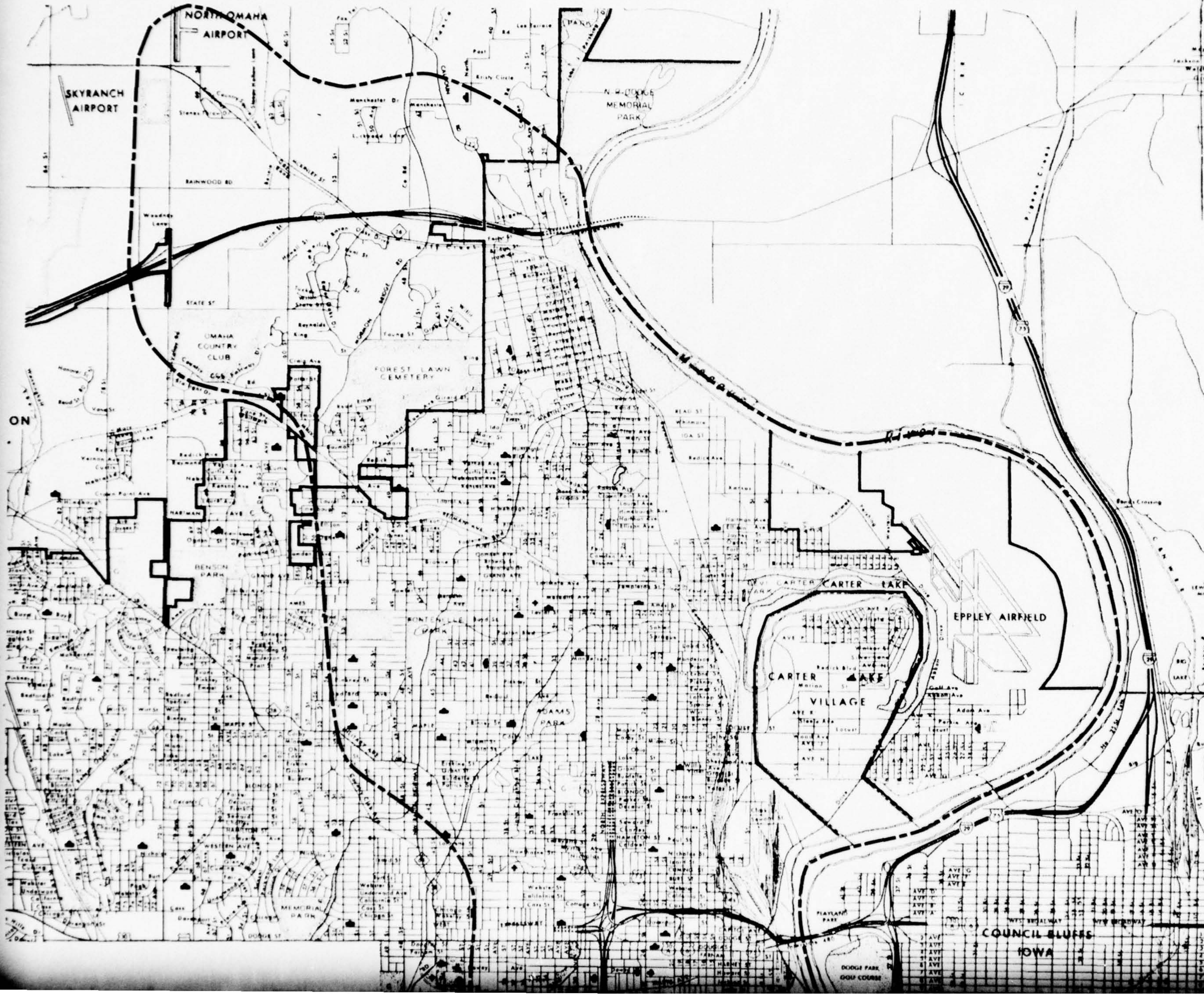
Direction for Further Study. The investigations described in this report, and the alternative plans that were formulated are essentially restricted to the combined sewer overflow problem. The alternative levels of effluent quality and design-storm recurrence intervals have not, as yet, been related to water quality impacts on the Missouri River. In addition, alternative plans were formulated and evaluated in this study without conducting field surveys, subsurface investigations, sampling programs, and detailed design of components. These investigations will be required as part of a feasibility study prior to a commitment for a construction investment.

Next steps should be in the direction of incorporation of the results of this investigation, and parallel investi-

gations concerned with other waste sources into the comprehensive planning program. A simulation model of the waste assimilative processes of the Missouri River and local tributaries should be a key element of the comprehensive planning program. With such a model, it will be possible to account for the interaction between various waste sources and to determine the required quality of effluent from each source in order to meet water quality standards in the River. The model also should provide a basis for optimizing investments in pollution control facilities toward the objective of maximum cost effectiveness.

The simulation model possibly may indicate that lower levels of treatment of combined sewer overflows and a reduction in design recurrence interval are acceptable without violation of water quality standards. If this is the case, consideration should be given to alternative concepts, such as in-system attenuation, that were rejected because they were inherently unable to meet the minimum performance and treatment criteria set for this study. Another promising area for further study is the computer simulation of the behavior of the aerated reservoirs included in the alternative plans for storage of combined sewer overflows. It is possible that the storage detention and aeration may produce an effluent that is acceptable for discharge to the river without additional treatment.

PLATES


NORTH OMAHA AIRPORT
SKYRANCH AIRPORT
N.P. DODGE MEMORIAL PARK
OMAHA COUNTRY CLUB
FOREST LAWN CEMETERY
BENSON PARK
ADAMS PARK
MEMORIAL PARK
CARTER LAKE
CARTER LAKE VILLAGE
EPPLEY AIRFIELD
DODGE PARK GOLF COURSE
COUNCIL BLUFFS
IOWA
ON

OUTLINE OF STUDY AREA

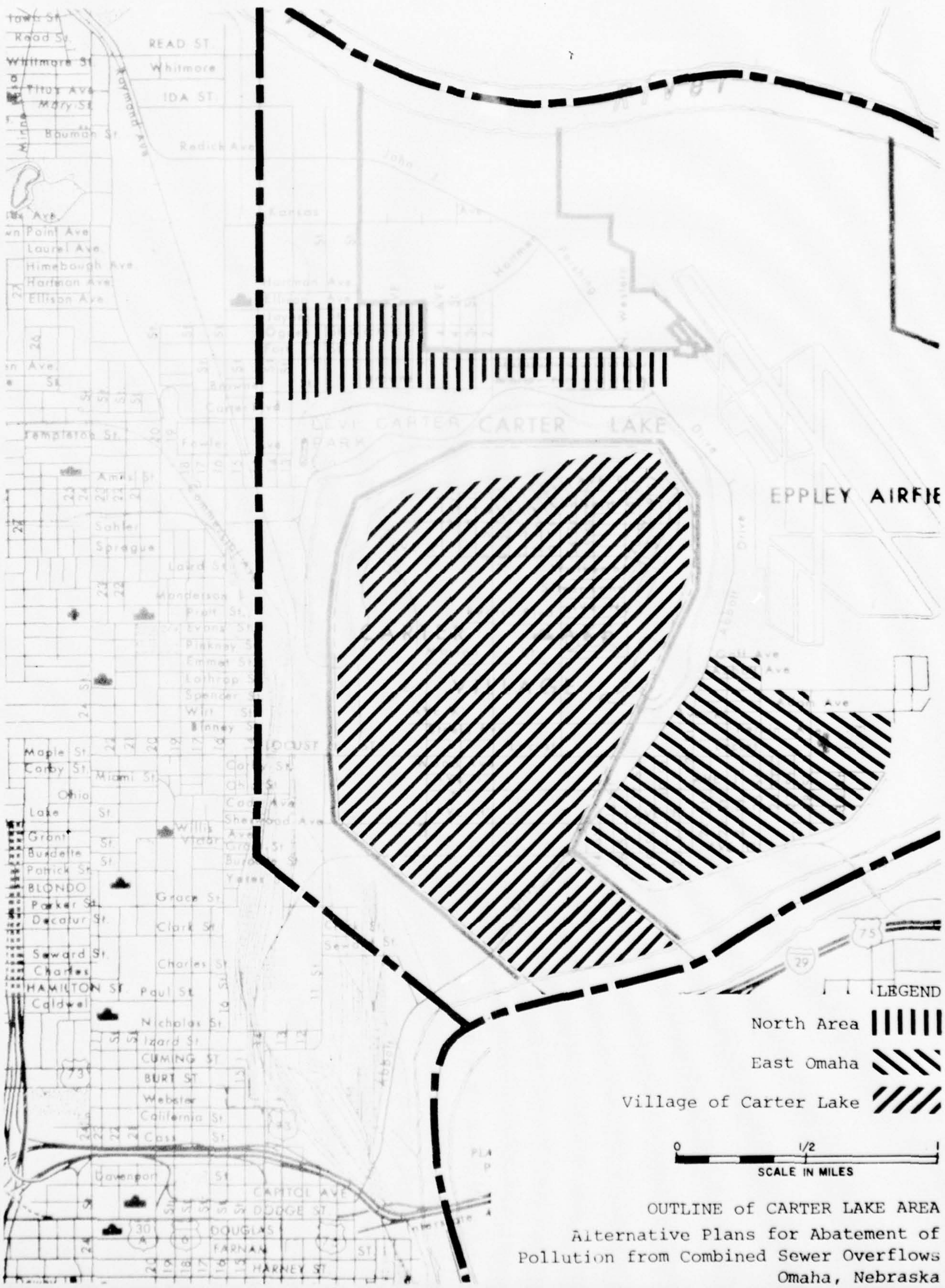
ALTERNATIVE PLANS FOR ABATEMENT OF

POLLUTION FROM COMBINED SEWER OVERFLOWS

OMAHA, NEBRASKA

PLATE 1

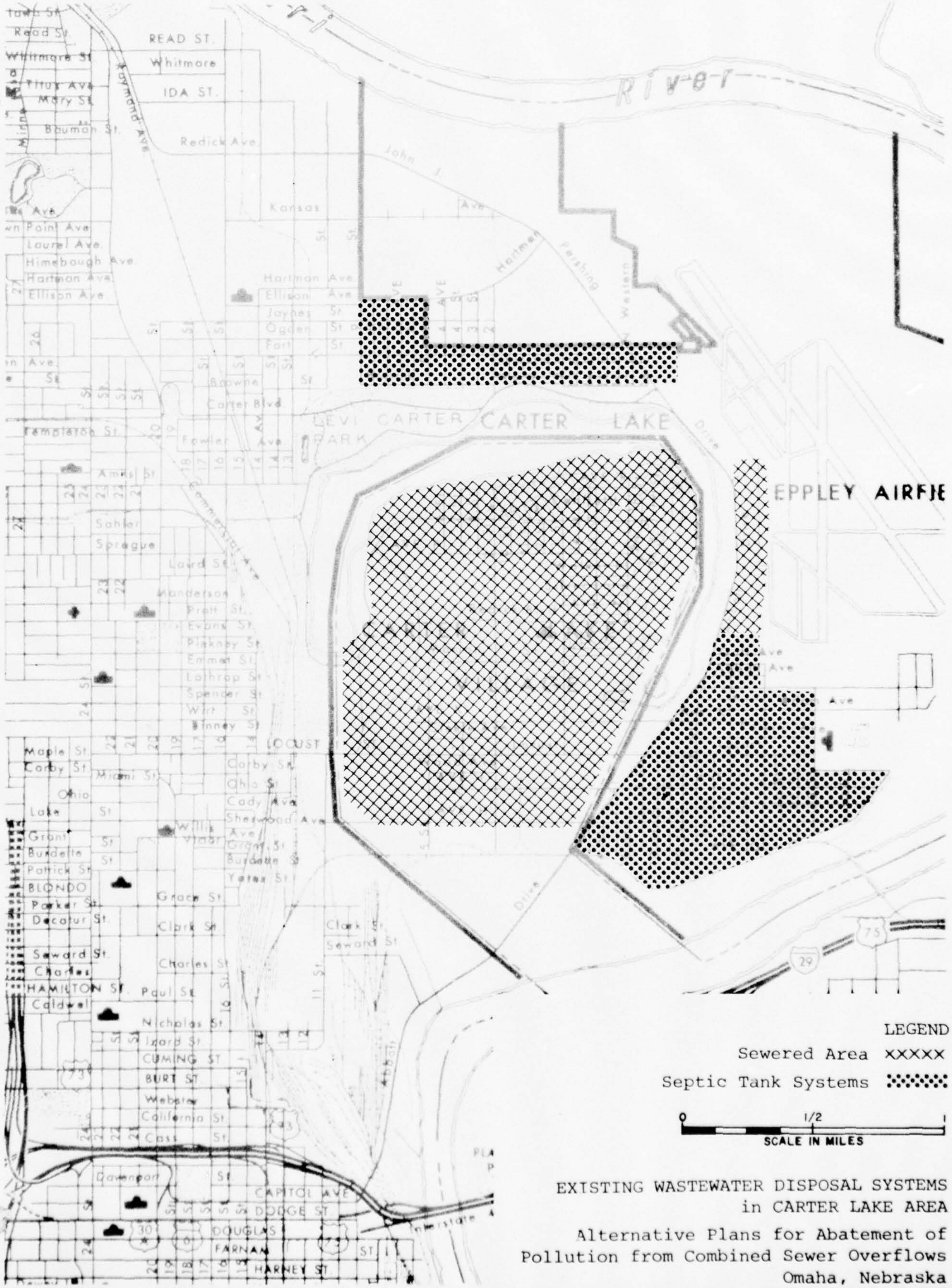
2



OUTLINE of CARTER LAKE AREA

Alternative Plans for Abatement of
Pollution from Combined Sewer Overflows
Omaha, Nebraska

PLATE 2



LEGEND

Sewered Area XXXXX

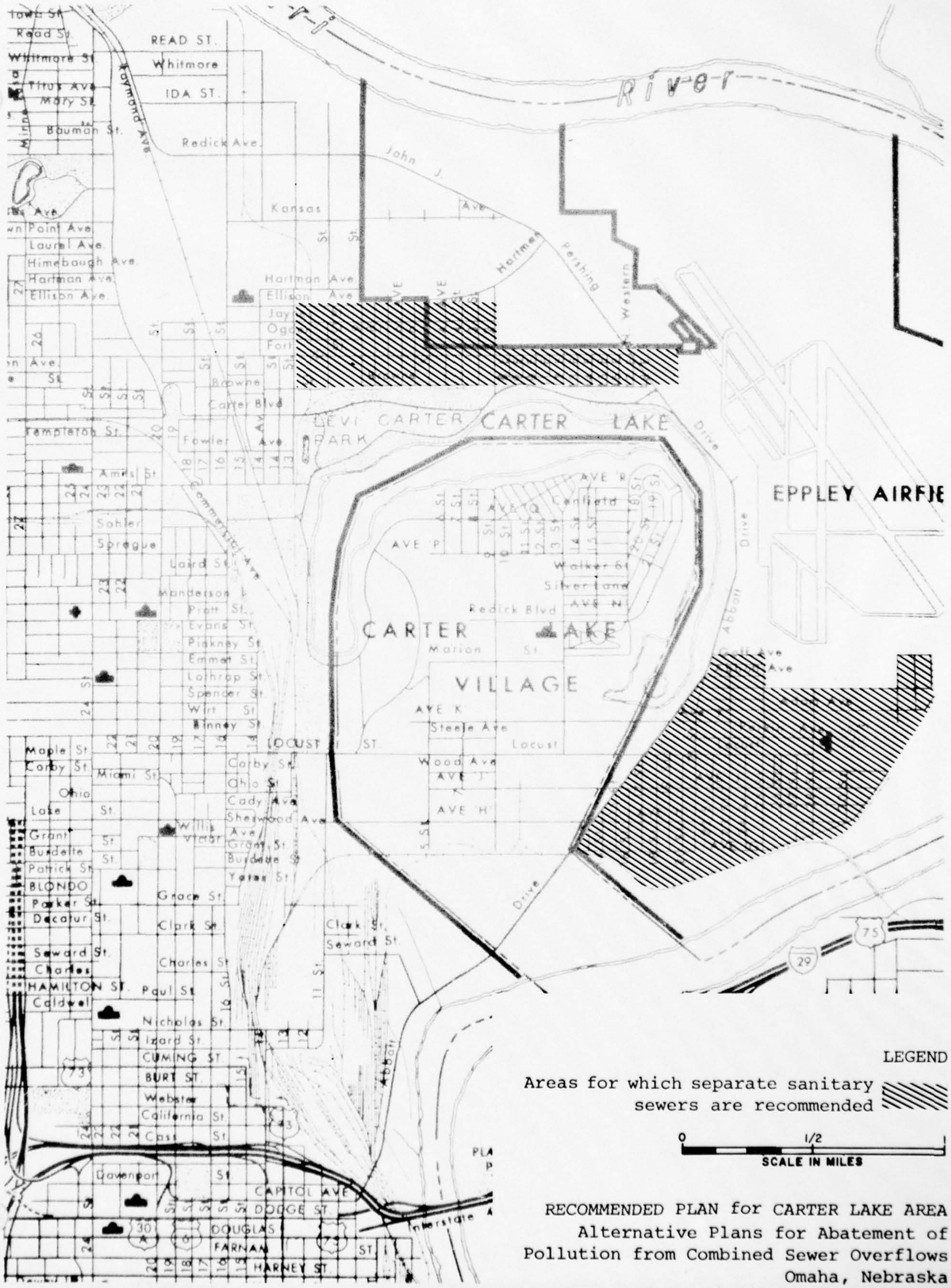
Septic Tank Systems

0 1/2 1

SCALE IN MILES

EXISTING WASTEWATER DISPOSAL SYSTEMS
in CARTER LAKE AREA

Alternative Plans for Abatement of
Pollution from Combined Sewer Overflows
Omaha, Nebraska

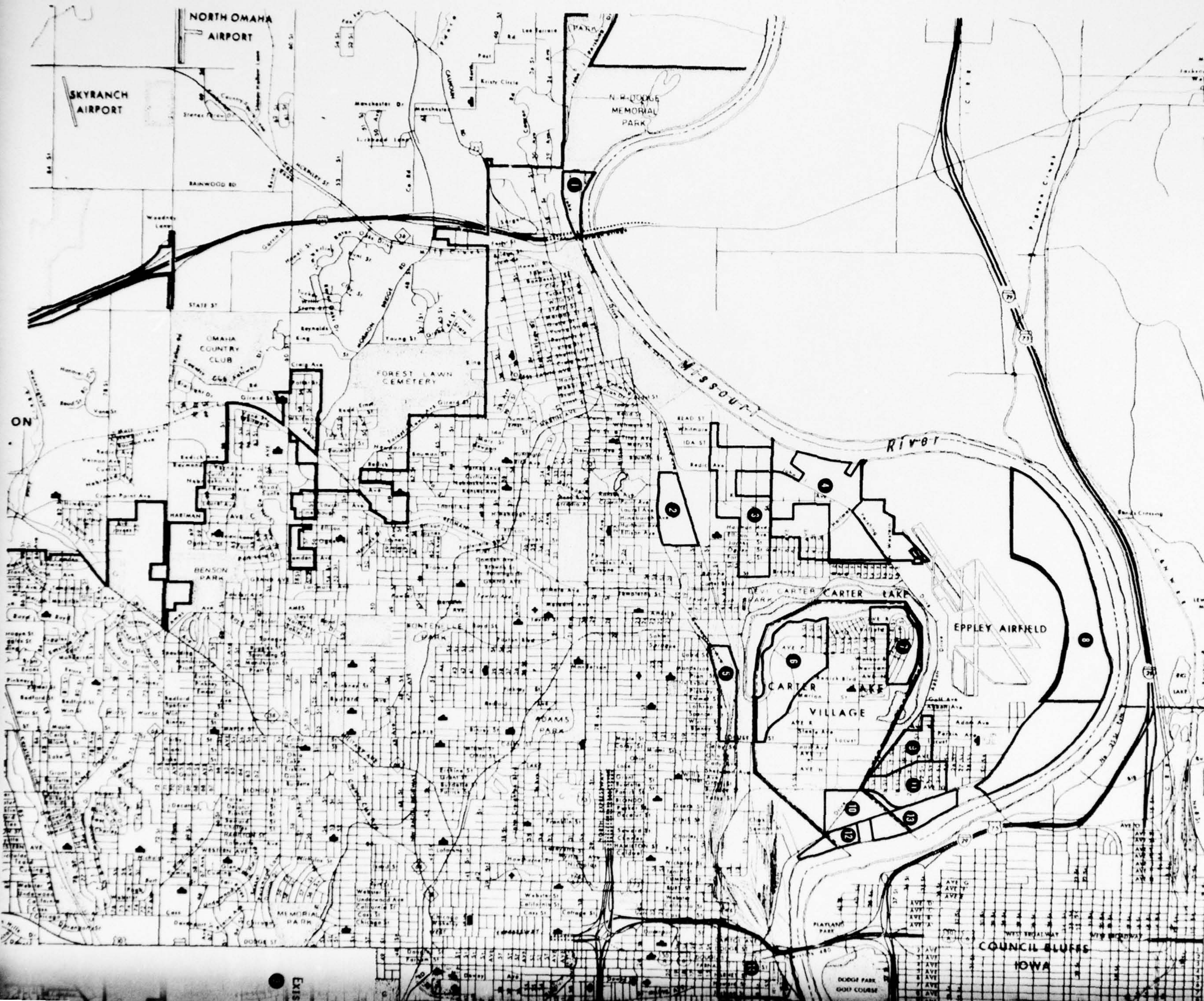


LEGEND

Areas for which separate sanitary sewers are recommended

0 1/2
SCALE IN MILES

RECOMMENDED PLAN for CARTER LAKE AREA
Alternative Plans for Abatement of
Pollution from Combined Sewer Overflows
Omaha, Nebraska



NORTH OMAHA AIRPORT
SKYRANCH AIRPORT
N.P. DODGE MEMORIAL PARK
OMAHA COUNTRY CLUB
FOREST LAWN CEMETERY
BENSON PARK
FONTENELLE PARK
ADAMS PARK
MEMORIAL PARK
Missouri River
EPPLEY AIRFIELD
CARTER LAKE
CARTER LAKE VILLAGE
COUNCIL BLUFFS IOWA
DODGE PARK GOLF COURSE
PLAYLAND PARK
1
2
3
4
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13

ON
Exis

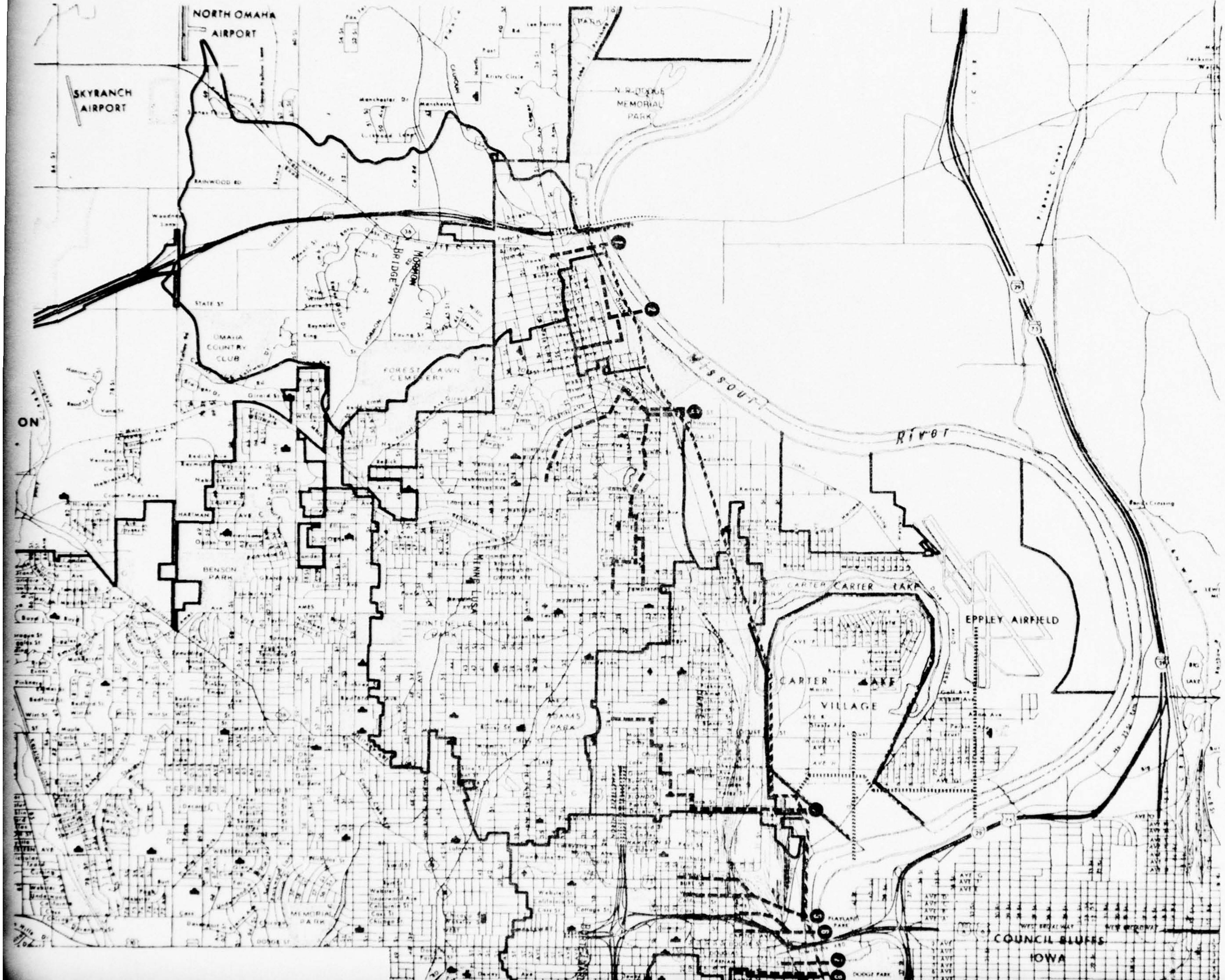
LOCATION OF RIVERFRONT DEVELOPMENT PROJECTS

ALTERNATIVE PLANS FOR ABATEMENT OF
POLLUTION FROM COMBINED SEWER OVERFLOWS

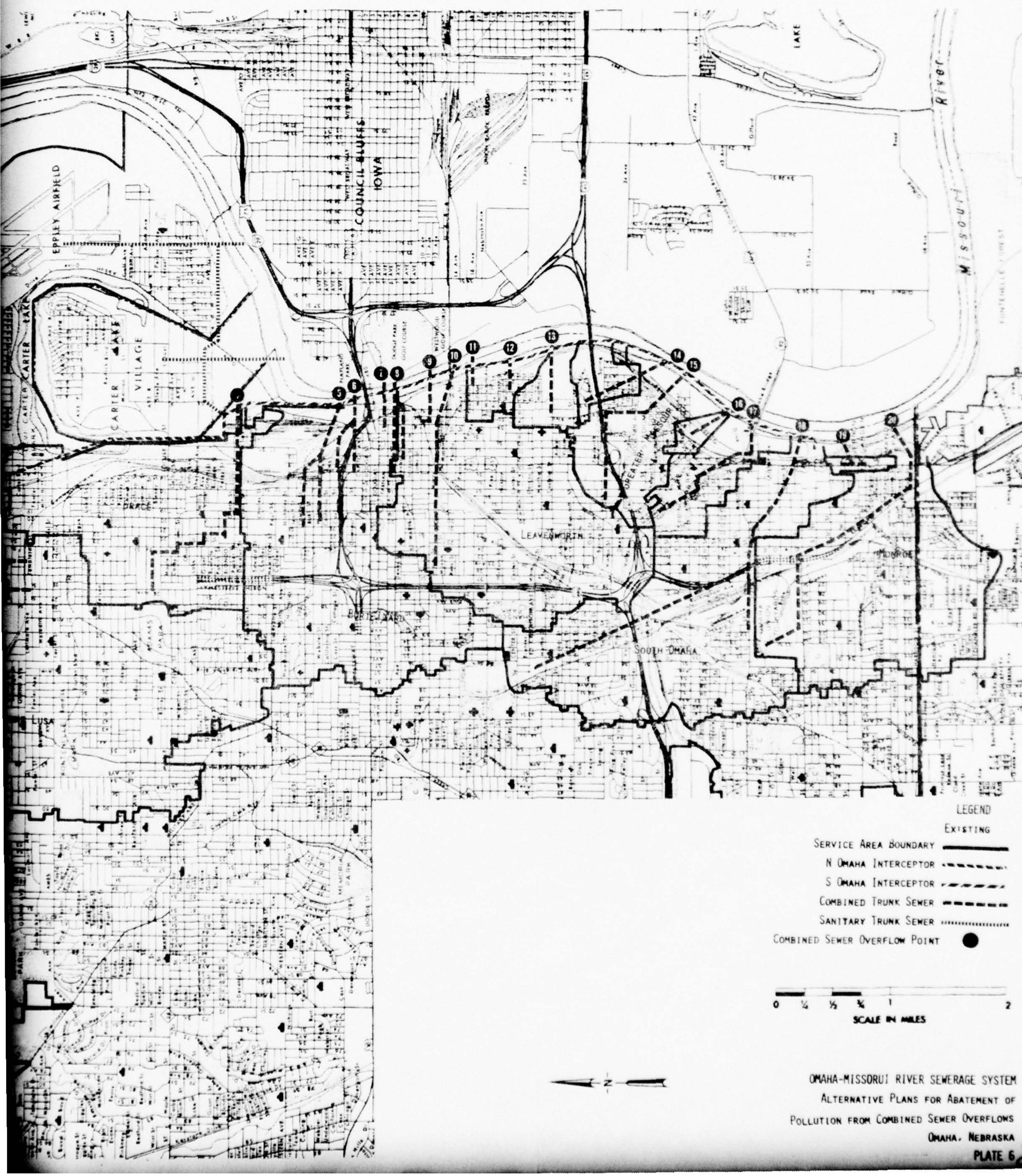
OMAHA, NEBRASKA

PLATE 5

2

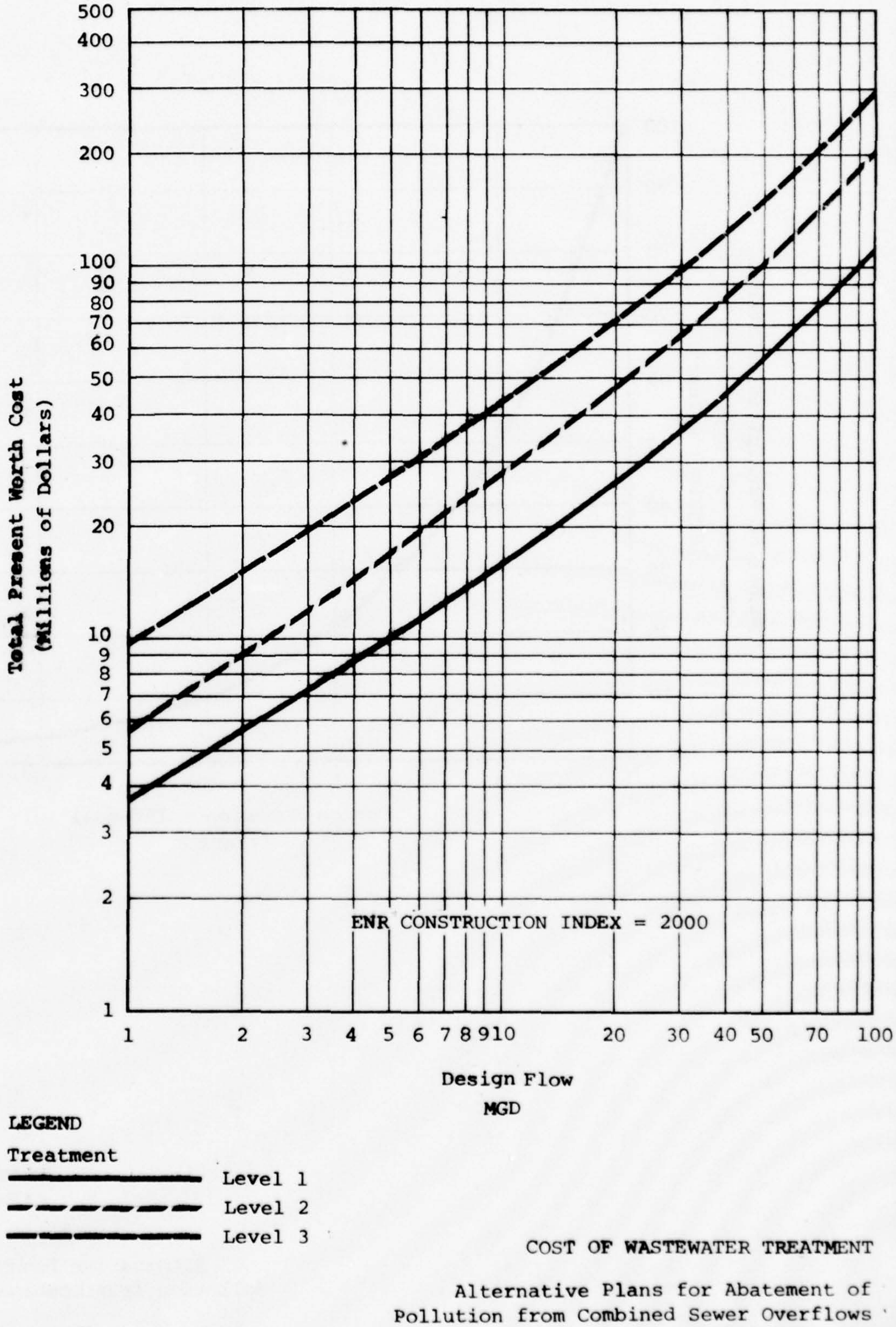


NORTH OMAHA AIRPORT
SKYRANCH AIRPORT
N.P. DODGE MEMORIAL PARK
MORMON BRIDGE
OMAHA COUNTRY CLUB
FOREST LAWN CEMETERY
MISSOURI
River
MINNE LUSA
FONTENELLE PARK
BENSON PARK
CARTER LAKE
EPPLEY AIRFIELD
CARTER LAKE VILLAGE
MEMORIAL PARK
DODGE PARK GOLF COURSE
COUNCIL BLUFFS
IOWA
ON



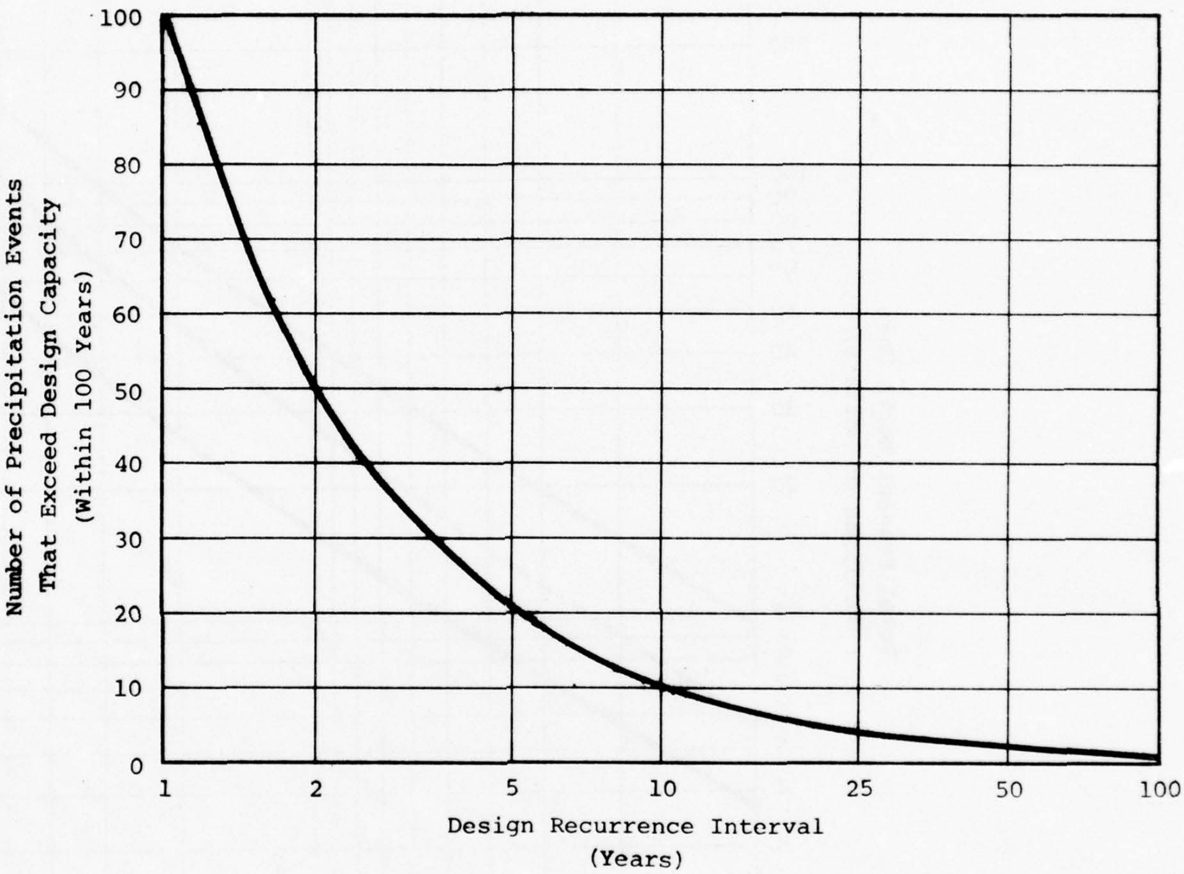
OMAHA-MISSORUI RIVER SEWERAGE SYSTEM
ALTERNATIVE PLANS FOR ABATEMENT OF
POLLUTION FROM COMBINED SEWER OVERFLOWS
OMAHA, NEBRASKA
PLATE 6 2

SEPARATE STORM SEWER AREAS
Alternative Plans for Abatement of
Pollution from Combined Sewer Overflows
Omaha, Nebraska



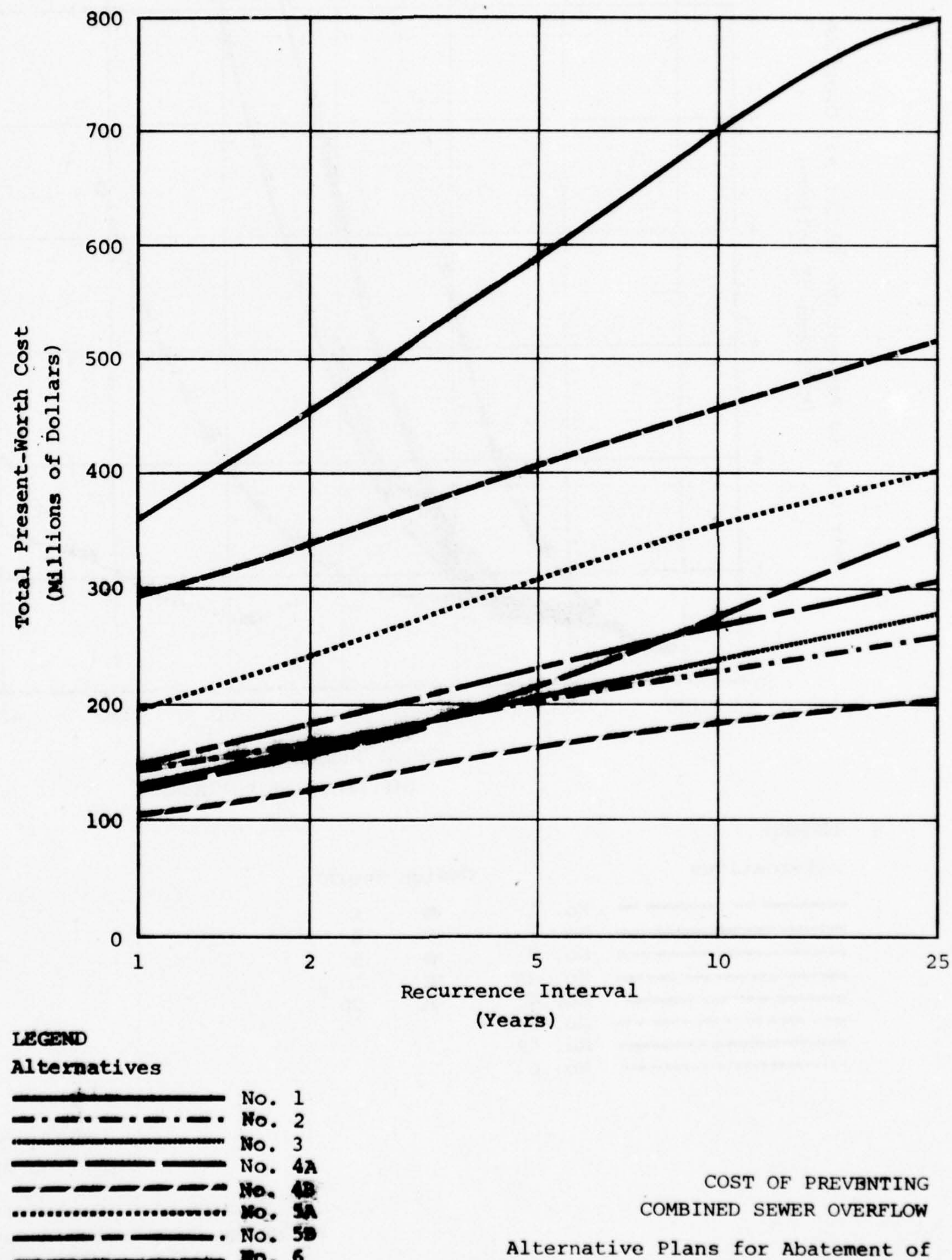
COST OF WASTEWATER TREATMENT

Alternative Plans for Abatement of
Pollution from Combined Sewer Overflows
Omaha, Nebraska



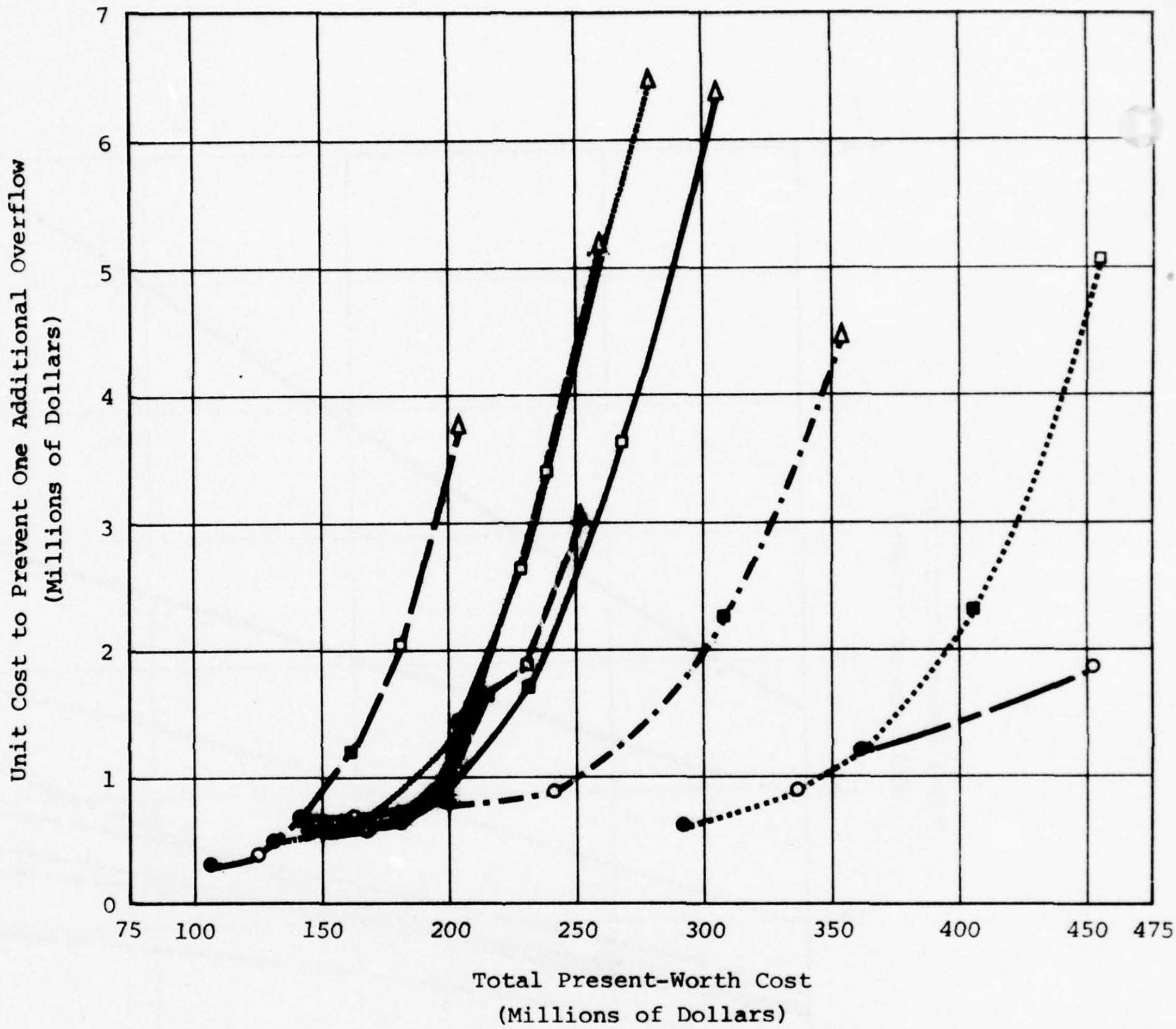
RELATIONSHIP OF DESIGN
CAPACITY TO NUMBER OF
TIMES OVERLOADED

Alternative Plans for Abatement of
Pollution from Combined Sewer Overflows
Omaha, Nebraska



COST OF PREVENTING
COMBINED SEWER OVERFLOW

Alternative Plans for Abatement of
Pollution from Combined Sewer Overflows
Omaha, Nebraska

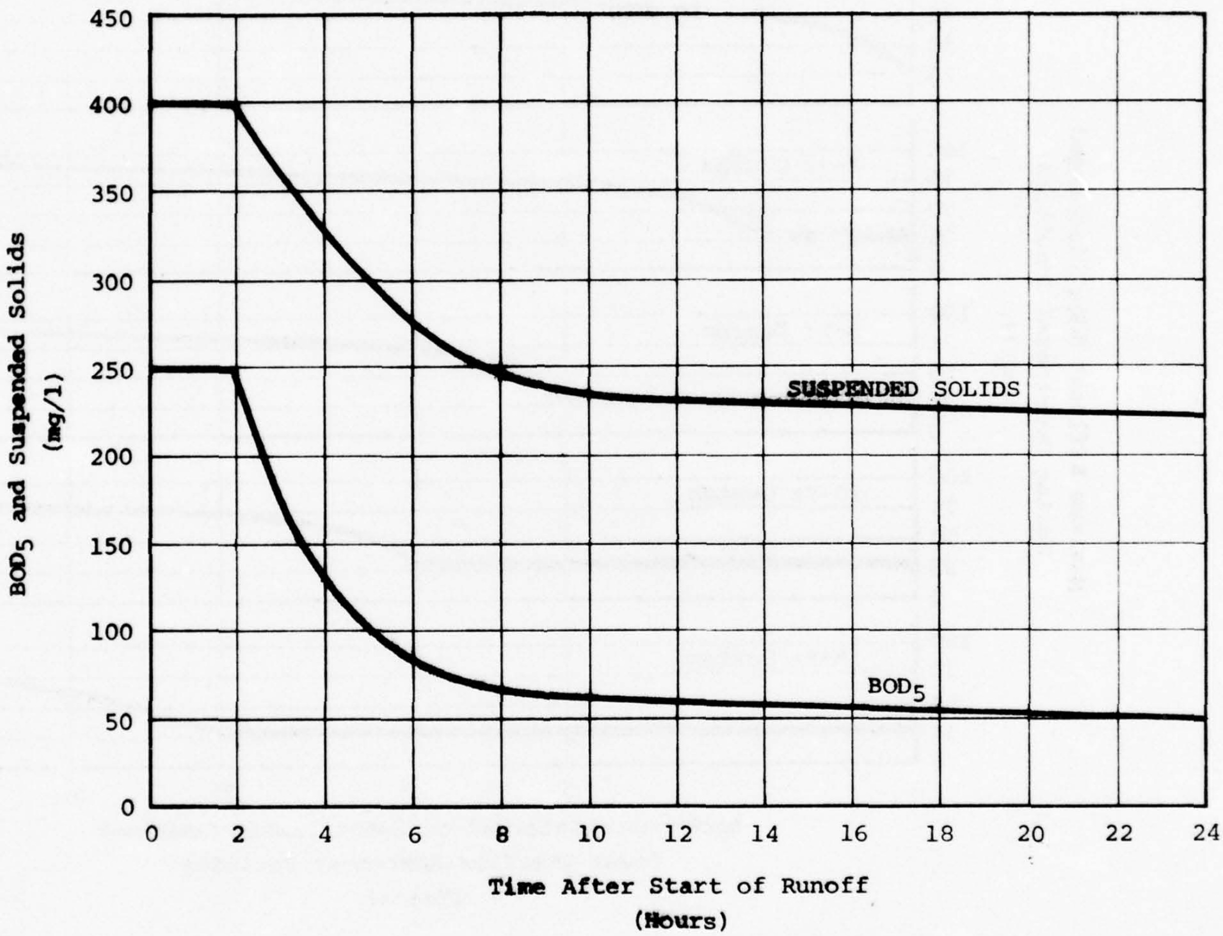


LEGEND

Alternatives	Design Years	
No. 1	●	1
No. 2	○	2
No. 3	■	5
No. 4A	□	10
No. 4B	△	25
No. 5A		
No. 5B		
No. 6		

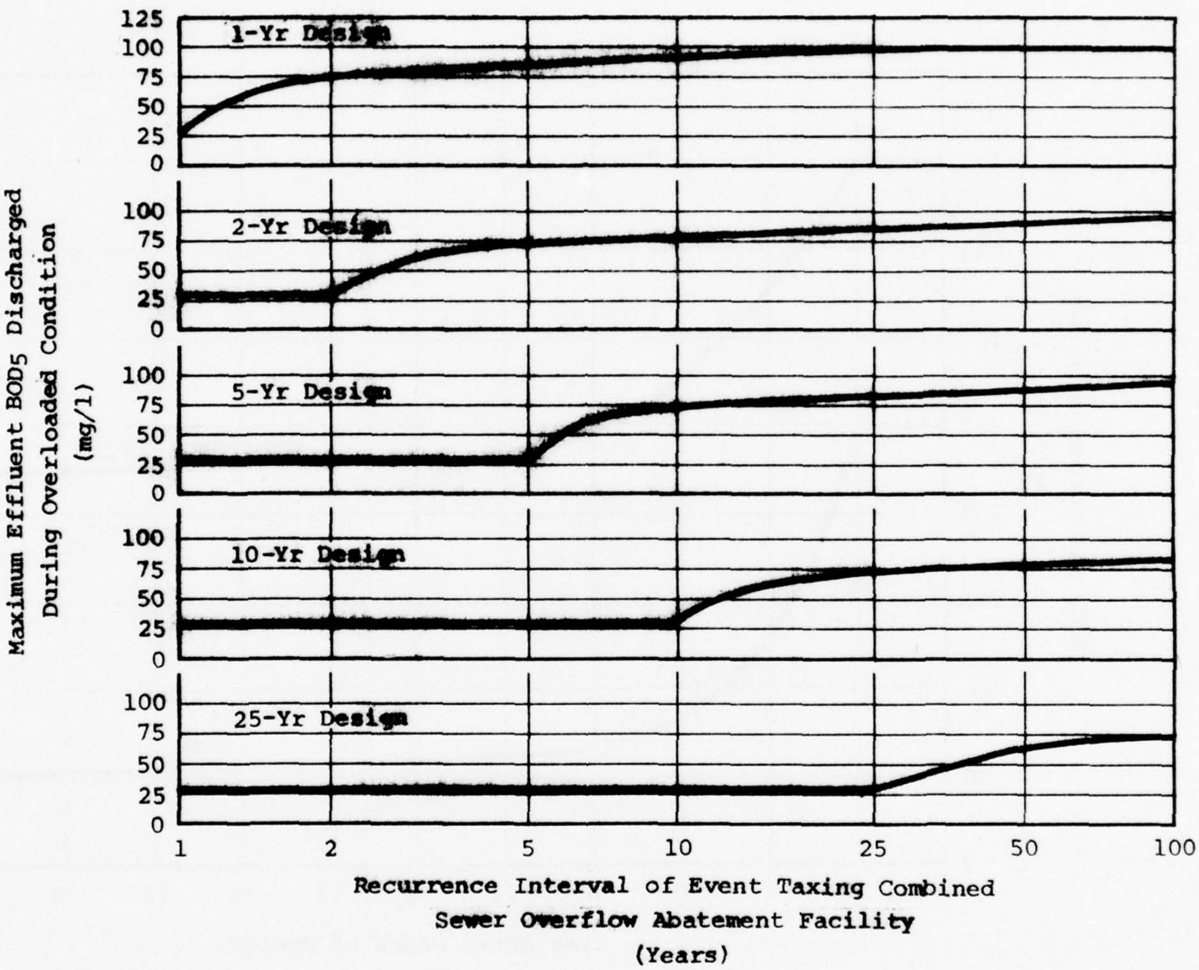
RELATIONSHIP BETWEEN
UNIT COST AND TOTAL COST

Alternative Plans for Abatement of
Pollution from Combined Sewer Overflows
Omaha, Nebraska



COMBINED SEWAGE OVERFLOW
STRENGTH DURING RUNOFF EVENT

Alternative Plans for Abatement of
Pollution from Combined Sewer Overflows
Omaha, Nebraska



PERFORMANCE OF ALTERNATIVE CONCEPT
UNDER OTHER THAN DESIGN CONDITIONS

Alternative Plans for Abatement of
Pollution from Combined Sewer Overflows
Omaha, Nebraska

COST OF ALTERNATIVES SIZED FOR 5 YEAR RECURRENCE INTERVAL

ALL COSTS IN MILLION OF DOLLARS

		System Components						Summary of Costs for Secondary Treatment at 7% Interest Rate				
		Conveyance		Storage		Treatment 4/						
		Constr Cost	Annual O&M	Constr Cost	Annual O&M	Constr Cost	Annual O&M	Total of Constr Costs	Land Costs	Total Annual O&M	Present Worth of Annual Costs 5/	Total Present Worth
1	Buried Storage at Outfalls	6.1	0.2	517.7	1.7	12.7	1.4	536.5	1.7	3.3	51.3	589.5
2	Diked Storage Along Levee	76.0	1.0	37.5	1.7	12.7	1.4	126.2	6.0	4.1	75.3	207.5
3	Upstream Retention	12.2	0.5	111.9	1.7	12.7	1.4	136.8	12.2	3.6	56.6	205.6
4 A	Deep Tunnel North to Ground Level Storage	128.5	0.3	18.1	1.7	12.7	1.4	159.3	2.2	3.4	53.1	214.6
4 B	Excavated Storage North-Deep tunnel South to Ground Storage	52.3	0.3	36.0	1.7	12.7	1.4	101.0	7.6	3.4	53.0	161.6
5 A	Deep Tunnel With Mined Storage	77.9	0.4	161.4	1.7	12.7	1.4	252.0	1.7	3.5	54.6	308.3
5 B	Excavated Storage North-Deep Tunnel to Mined Storage South	41.6	0.3	120.5	1.7	12.7	1.4	174.8	5.5	3.4	52.9	233.2
6	Flow-Through Treatment With Storage at Outfall	8.4	0.3	336.1	1.1	31.1	3.0	375.6	0.3	17.0	47.3	423.2
7	Sewer Separation 1/	539.3	-	-	-	-	-	539.3	-	-	-	539.3
A	In-system Attenuation Devices 2/	-	-	70.2	0.3	12.7	1.4	82.9	1.7	1.7	39.5	123.4
B	Deep Tunnel to the Papillon Creek STP 3/	96.0	0.1	110.8	1.9	3/	3/	206.8	-	2.0	29.8	236.6
C	Flow-Through Treatment for "First Flush"	-	-	-	-	278.9	0.3	278.9	2.4	0.3	55.5	336.8

1/ For storms of any recurrence intervals

2/ Capable of reducing number of overflows to several times per year

3/ Does not include treatment - regional concept

4/ Costs for higher levels of treatment can be obtained from Plate 8

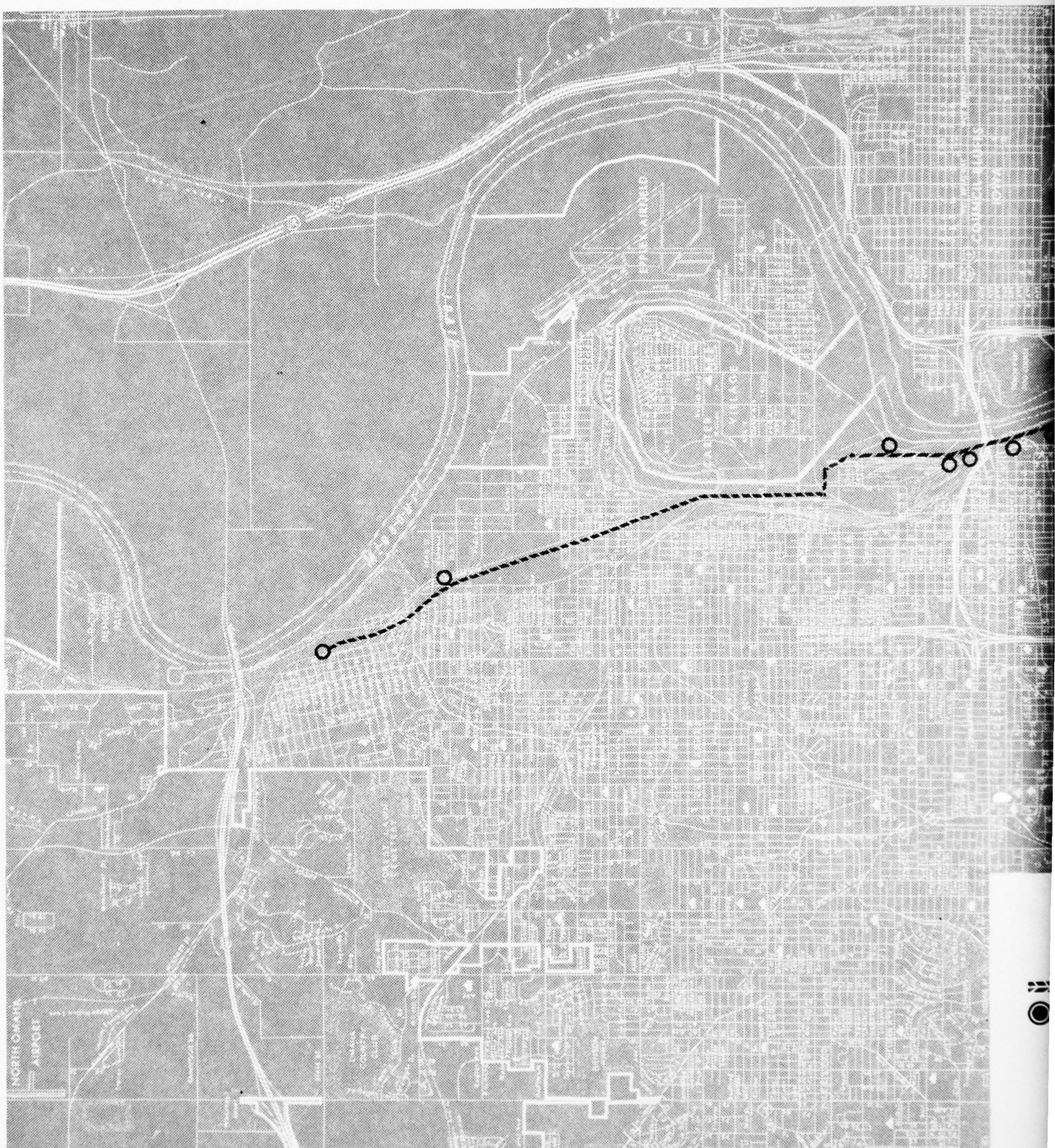
5/ Includes costs for replacement

SUBJECTIVE EVALUATION

OF ALTERNATIVES

		Aes-thetic Effects	Disrup-tive Effects	Likelihood of Public Acceptance	Treatment Attain-Ability	Flexi-bility for Staging	Site Avail-ability	Maintenance and Operation	Redundancy Effect[1/]	Energy Use
1	Buried Storage at Outfalls	Good	Low	Good	Good	Good	Poor	Poor	Excellent	Good
2	Diked Storage Along Levee	Poor	Moderate	Fair	Good	Fair	Fair	Fair	Excellent	Poor
3.	Up-stream Retention	Poor	High	Poor	Fair	Good	Poor	Poor	Poor	Good
4A	Deep Tunnel north to Ground Level Storage	Fair	Low	Good	Good	Poor	Good	Good	Excellent	Fair
4B	Excavated Storage North-Deep Tunnel South to Ground Storage	Fair	Moderate	Fair	Good	Poor	Good	Fair	Good	Fair
5A	Deep Tunnel with Mined Storage	Good	Low	Excellent	Good	Poor	Good	Good	Excellent	Poor
5B	Excavated Storage North-Deep Tunnel to Mined Storage South	Fair	Moderate	Fair	Good	Poor	Good	Fair	Good	Fair
6	Flow-Through Treatment With Storage at Outfall	Poor	Moderate	Fair	Fair	Good	Fair	Poor	Poor	Good
7	Sewer Separation	Good	High	Poor	Excellent	Good	Good	Good	Poor	Good
A	In-System Attenuation Devices	Good	Moderate	Good	Poor	Good	Good	Poor	Poor	Good
B	Deep Tunnel to the Papillon Creek STP	Good	Low	Good	Excellent	Poor	Good	Good	Excellent	Poor
C	Flow-through Treatment for "First Flush"	Poor	Moderate	Fair	Poor	Good	Fair	Poor	Poor	Good

1/ Capability of preventing dry-weather overflows resulting from mechanical failure of pumping facility on interceptor system or breakdown of sewage treatment plant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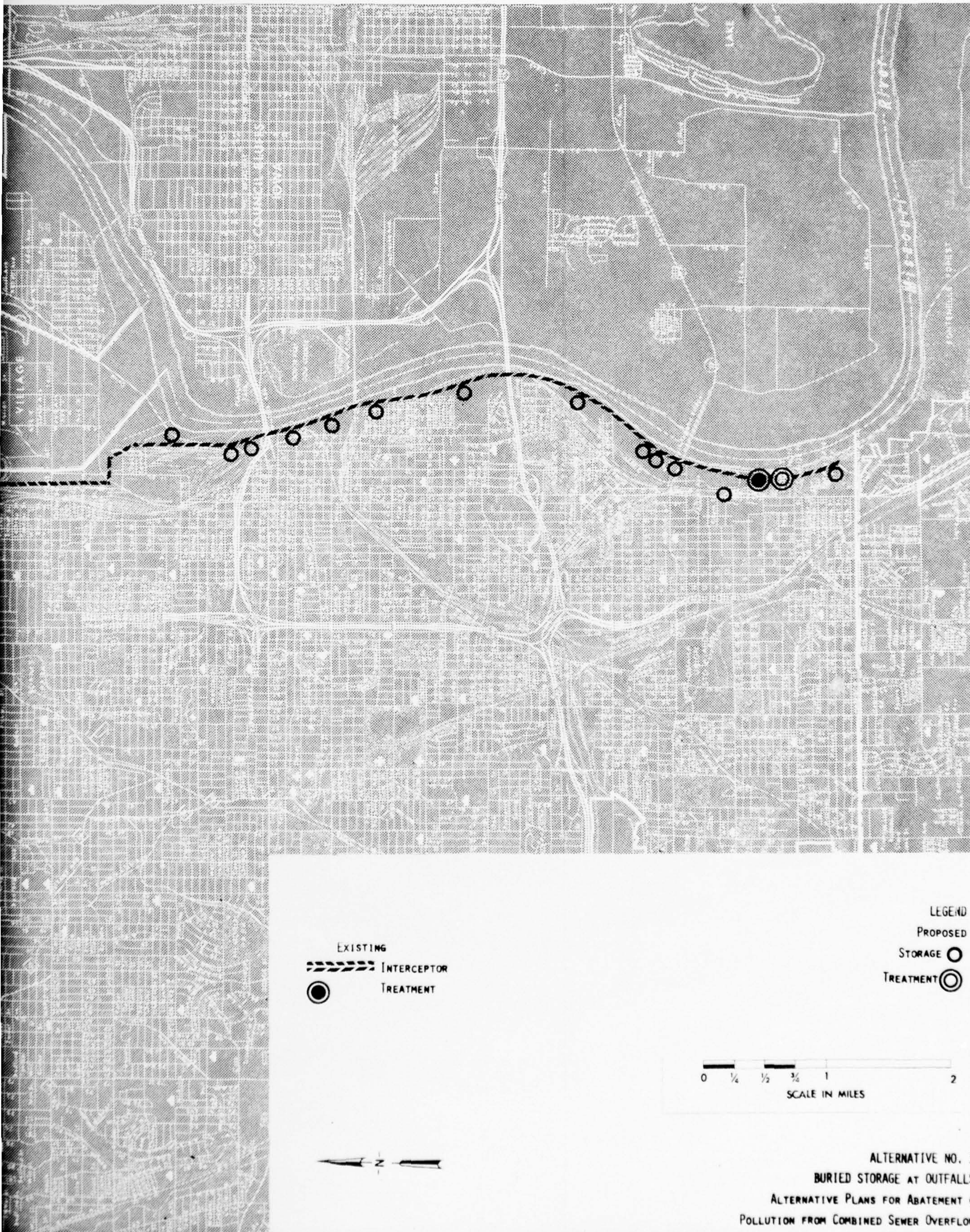
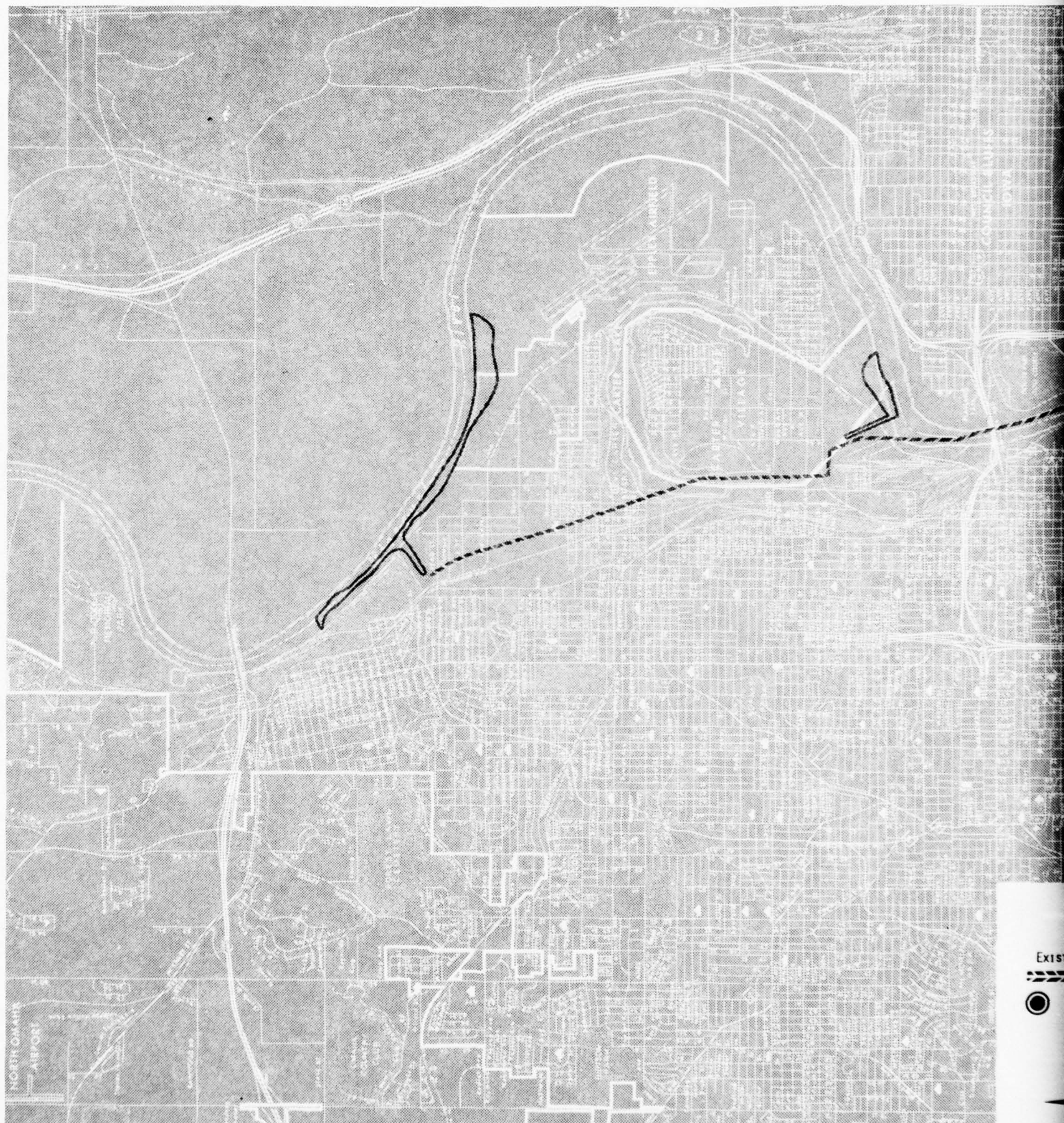
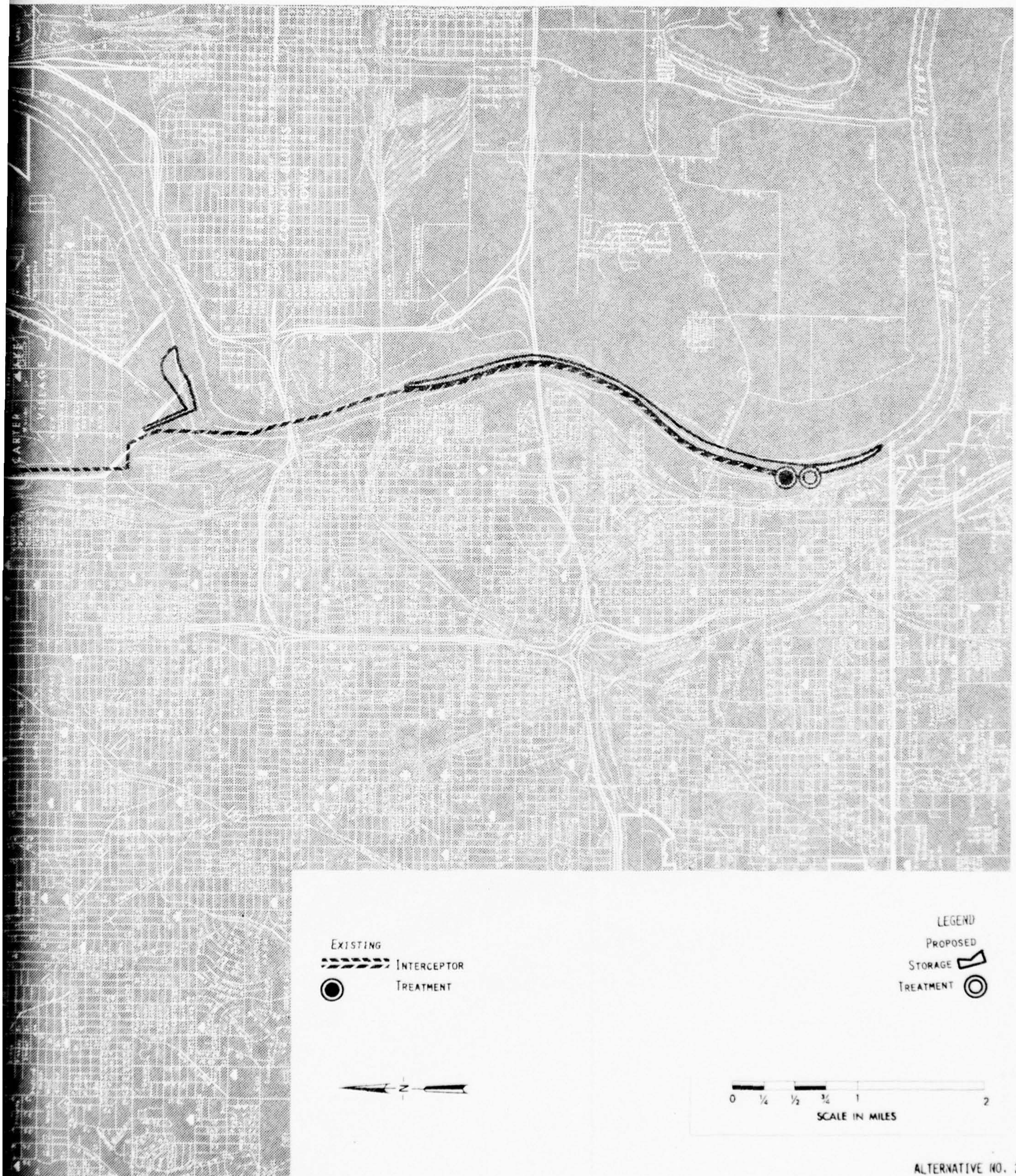


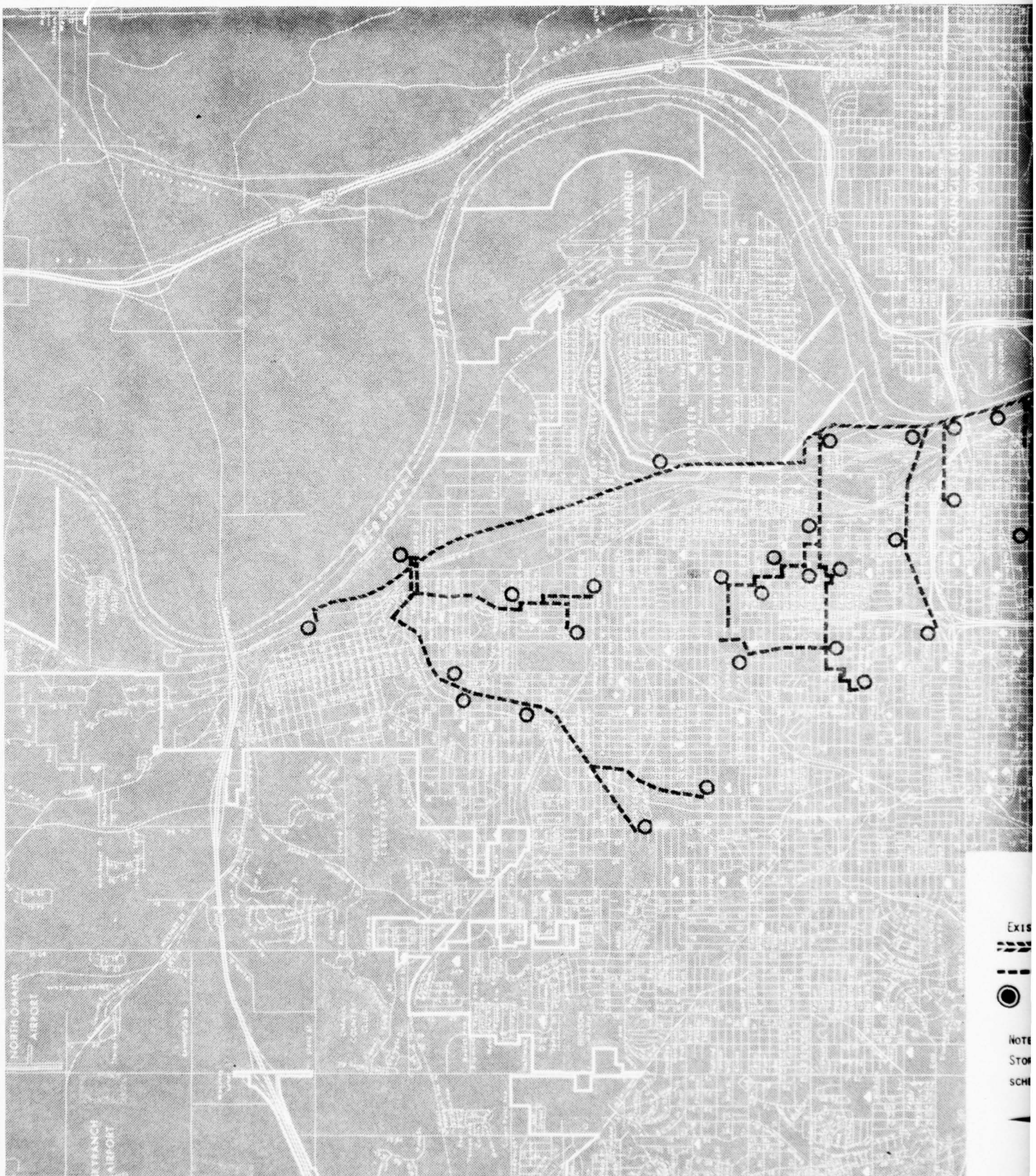
ALTERNATIVE NO. 1
BURIED STORAGE AT OUTFALLS
ALTERNATIVE PLANS FOR ABATEMENT OF
POLLUTION FROM COMBINED SEWER OVERFLOWS
OMAHA, NEBRASKA
PLATE 16





ALTERNATIVE NO. 2
DIKED STORAGE ALONG LEVEE
ALTERNATIVE PLANS FOR ABATEMENT OF
POLLUTION FROM COMBINED SEWER OVERFLOWS
OMAHA, NEBRASKA
PLATE 17

2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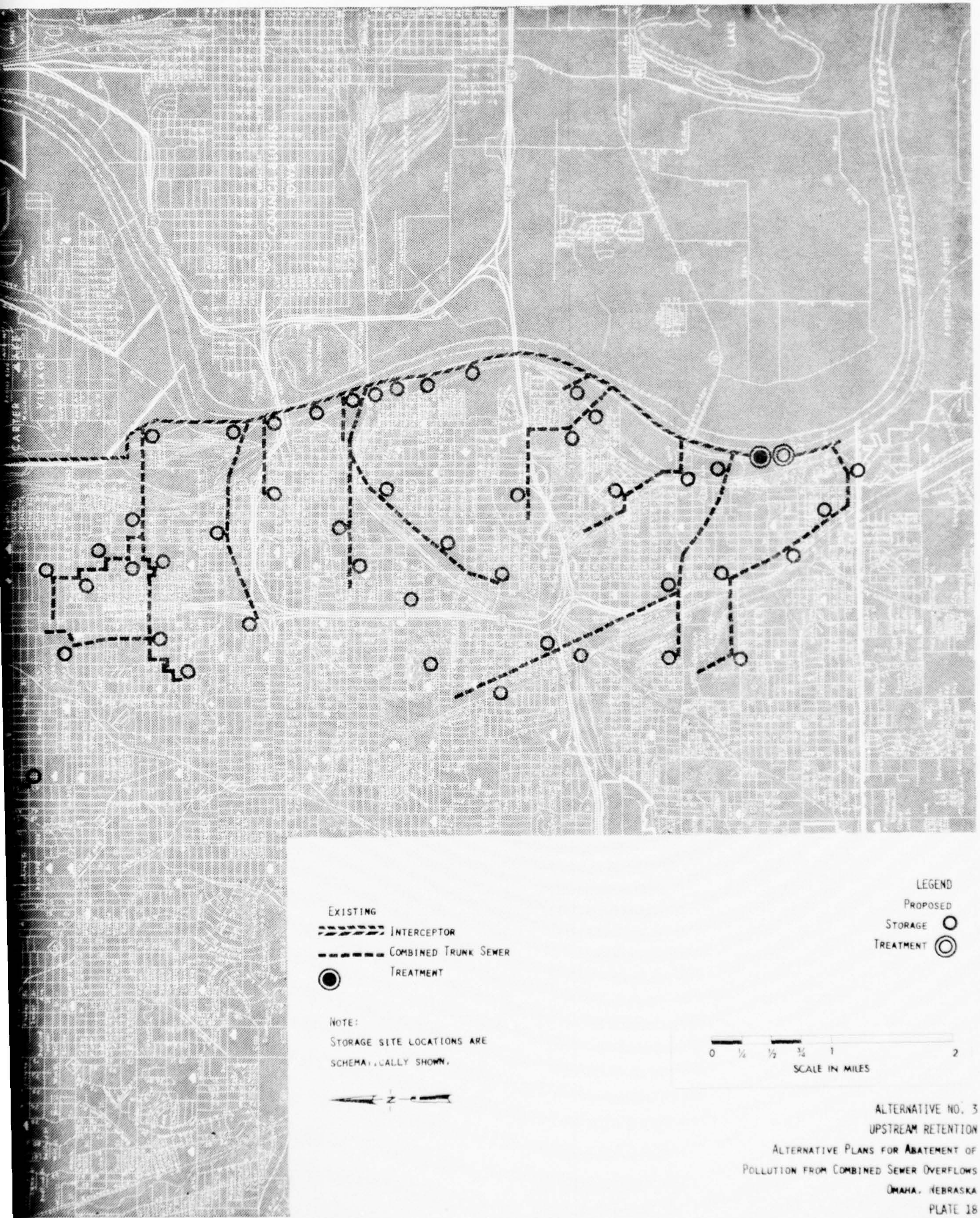
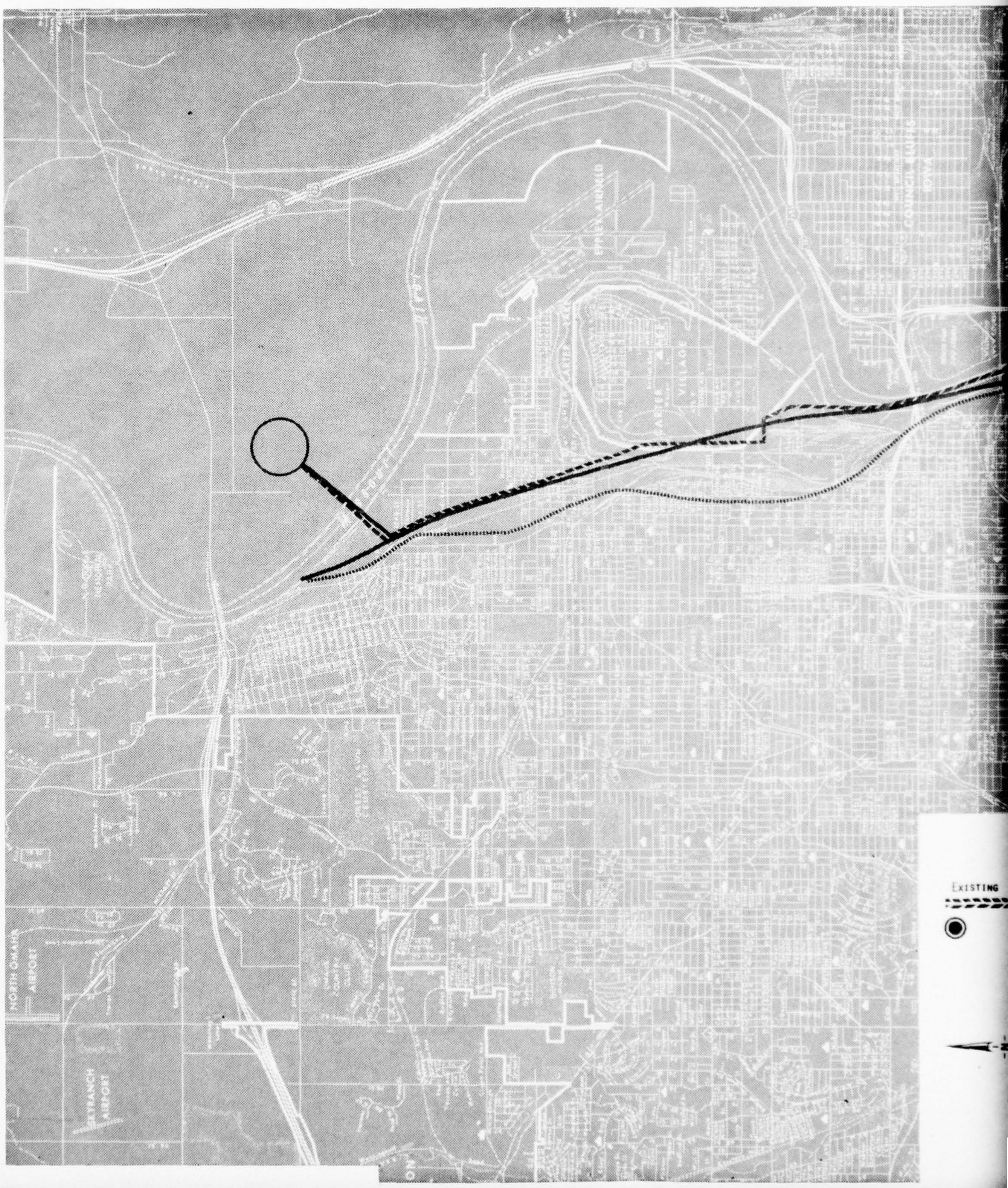
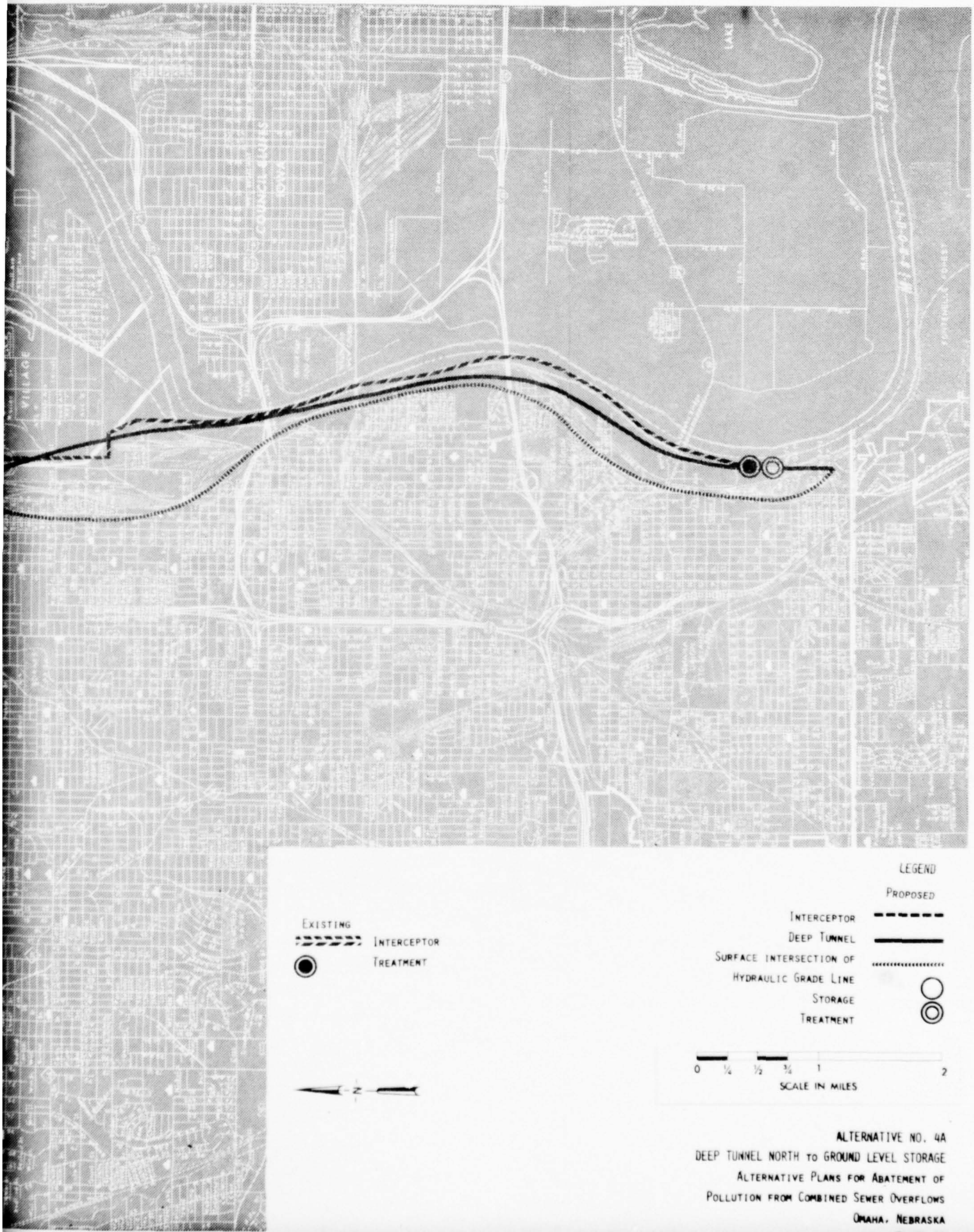


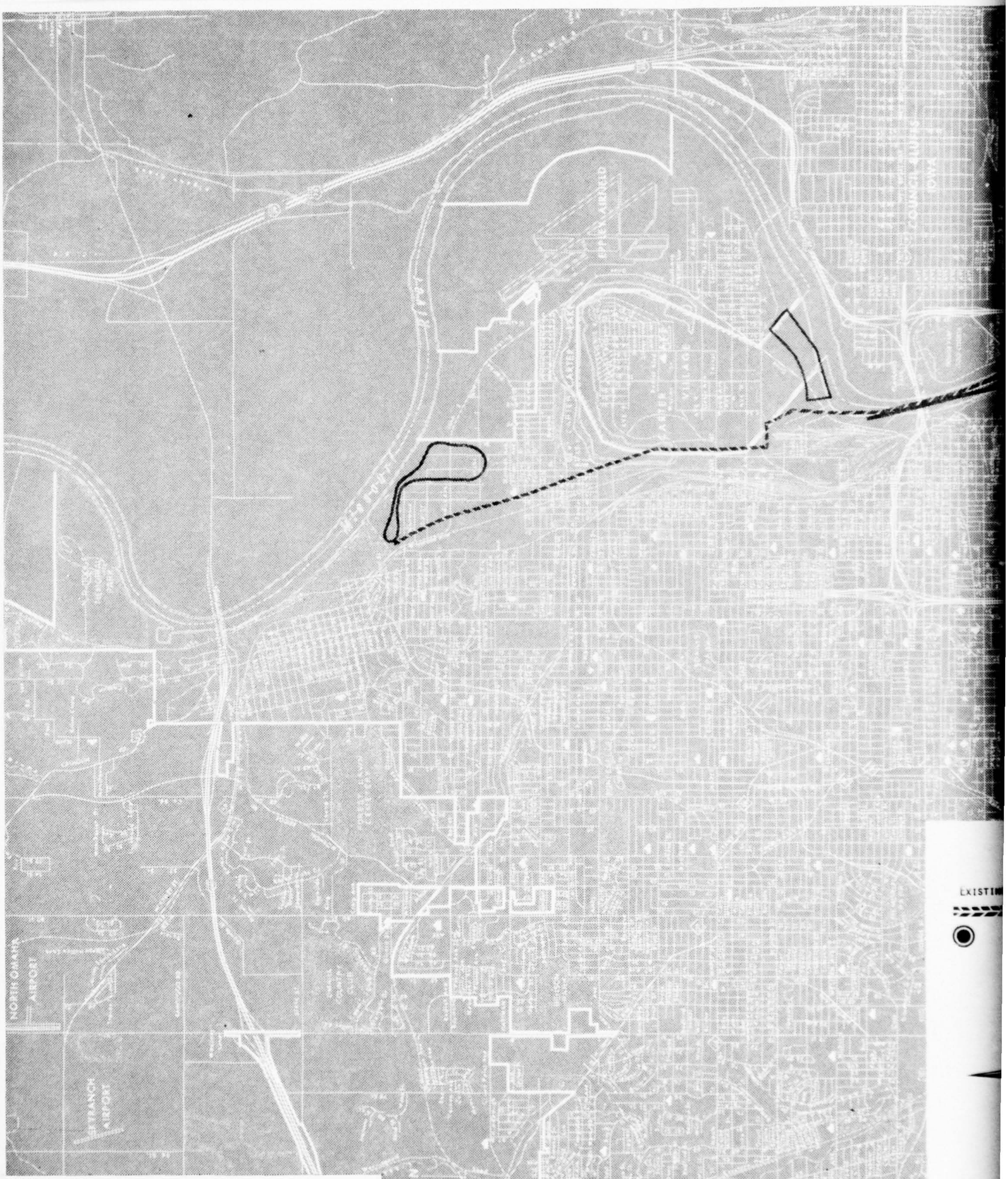
ALTERNATIVE NO. 3
UPSTREAM RETENTION
ALTERNATIVE PLANS FOR ABATEMENT OF
POLLUTION FROM COMBINED SEWER OVERFLOWS
OMAHA, NEBRASKA
PLATE 18

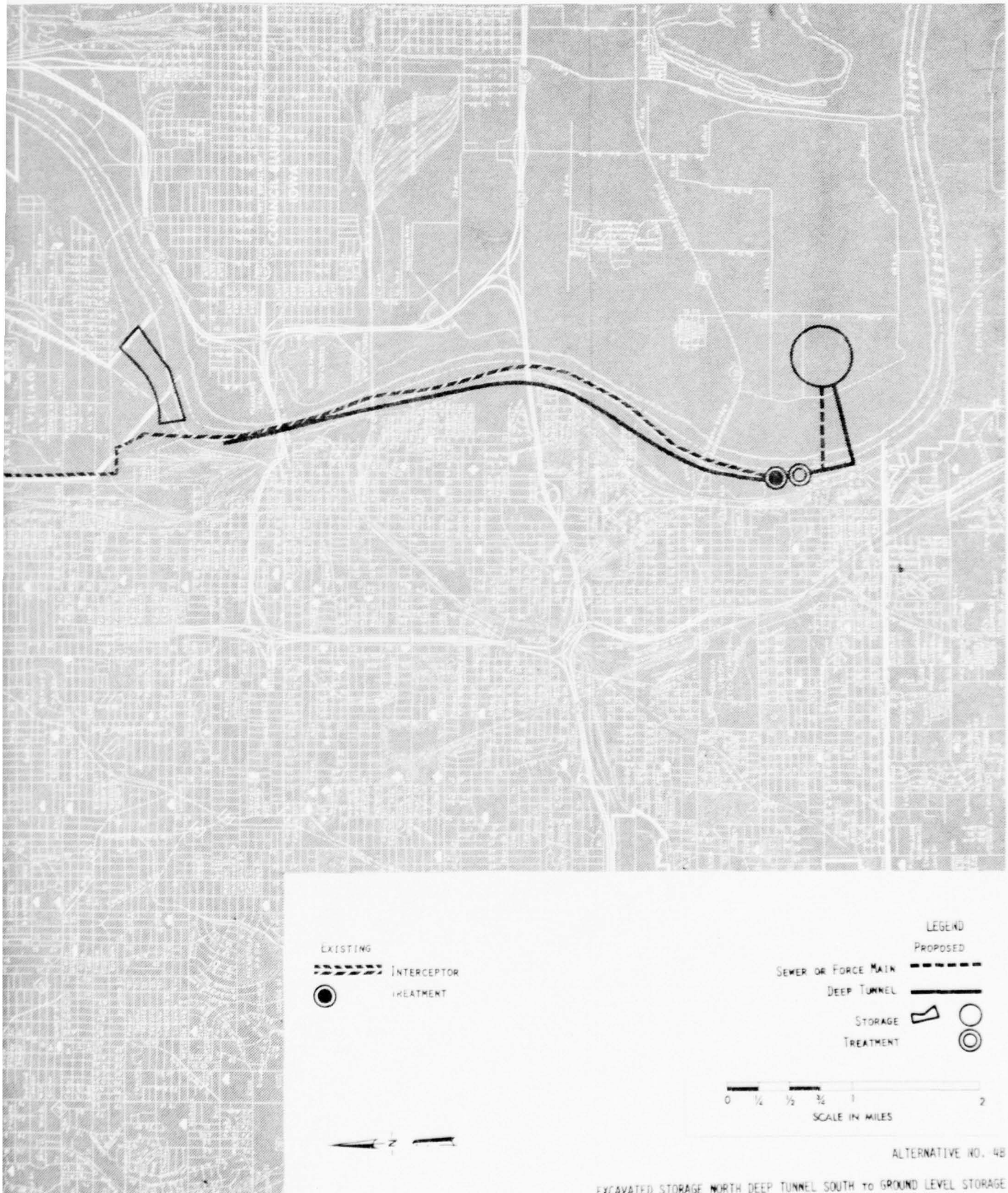
2


EPPLEY AIRFIELD
NORTH OMAHA AIRPORT
EXISTING



ALTERNATIVE NO. 4A
DEEP TUNNEL NORTH TO GROUND LEVEL STORAGE
ALTERNATIVE PLANS FOR ABATEMENT OF
POLLUTION FROM COMBINED SEWER OVERFLOWS
OMAHA, NEBRASKA
PLATE 19


NORTH OMAHA AIRPORT
COUNCIL BLUFFS
IOWA
EXISTI



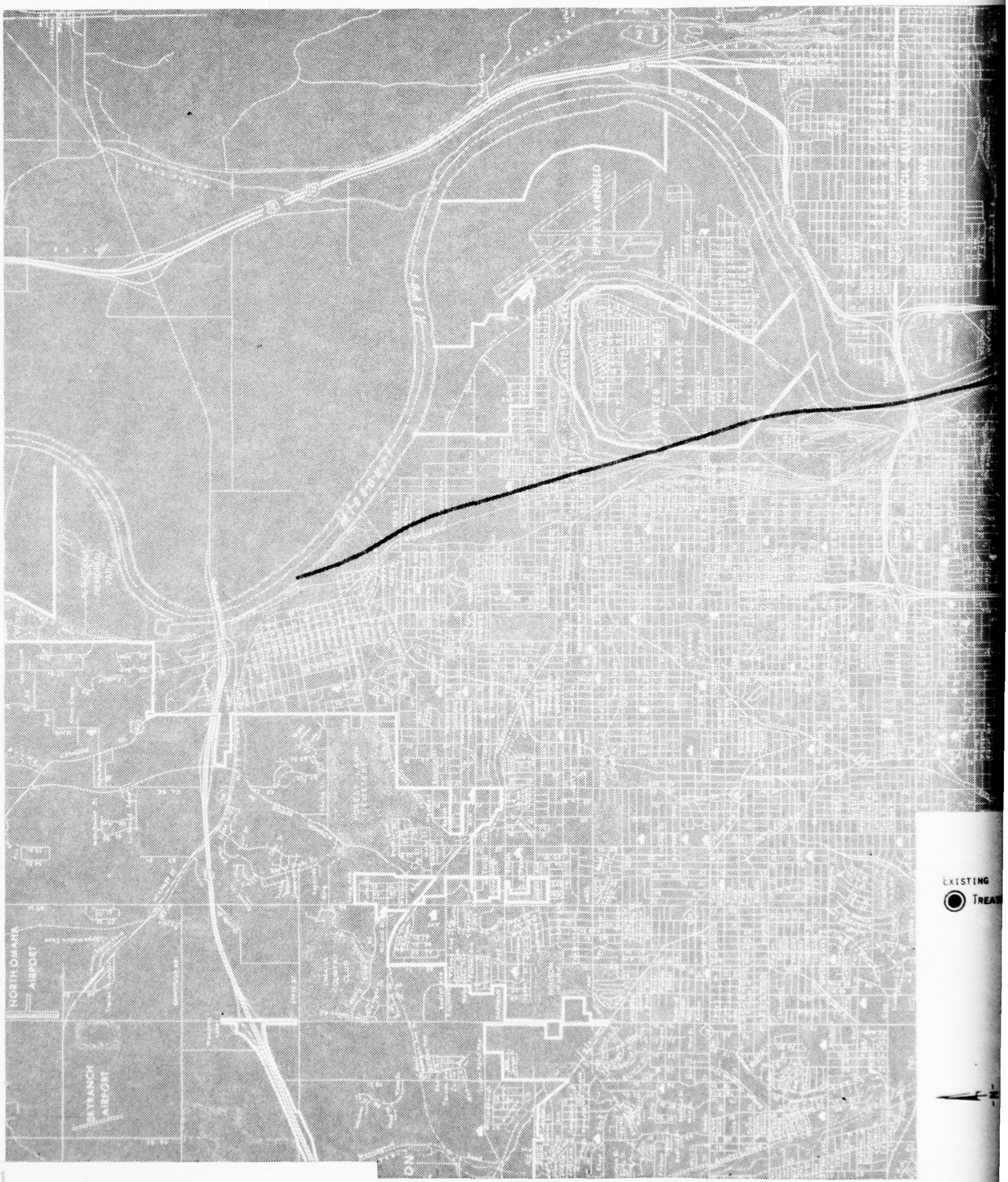
ALTERNATIVE NO. 4B

EXCAVATED STORAGE NORTH DEEP TUNNEL SOUTH TO GROUND LEVEL STORAGE

ALTERNATIVE PLANS FOR ABATEMENT OF POLLUTION FROM COMBINED SEWER OVERFLOWS

OMAHA, NEBRASKA

PLATE 20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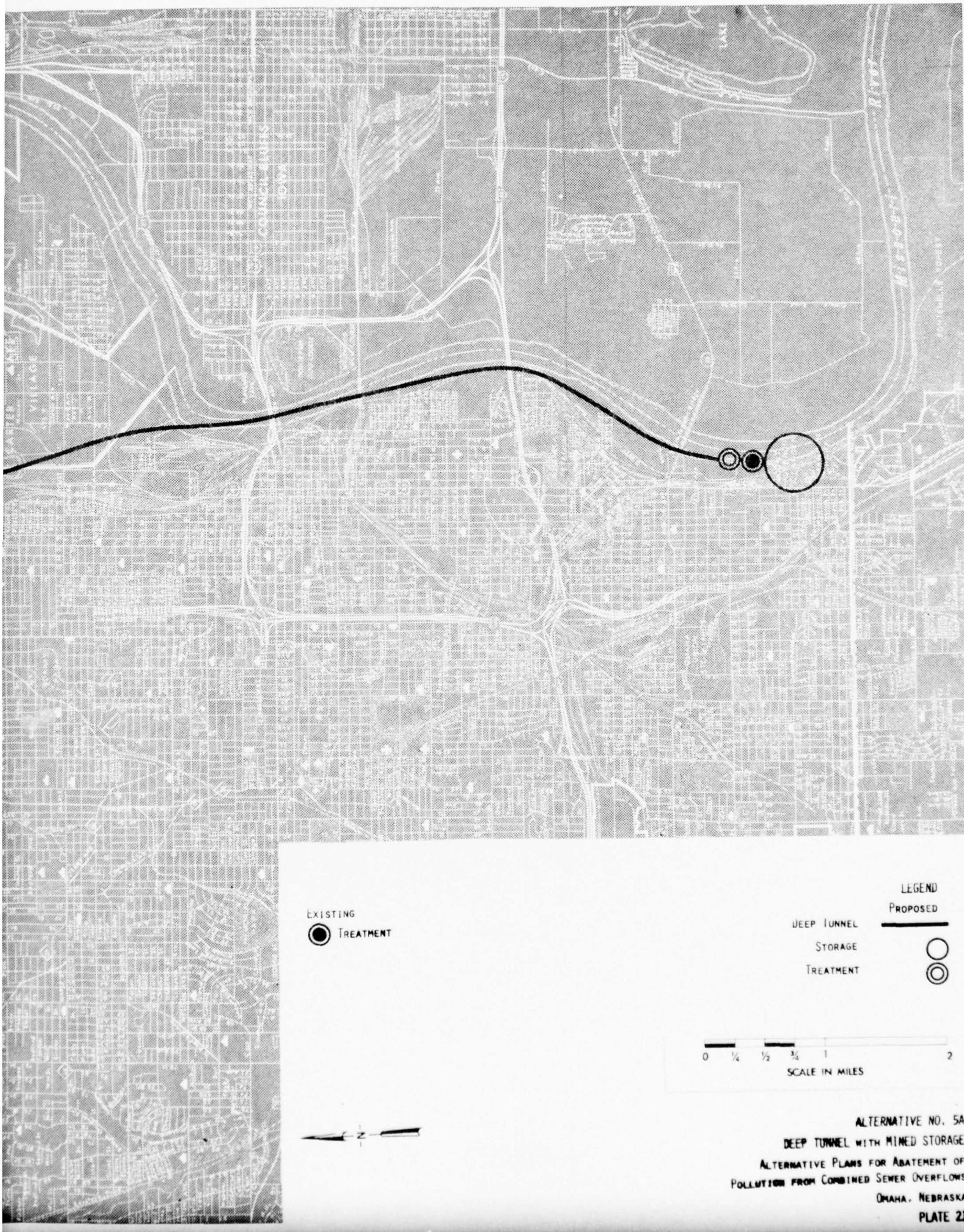
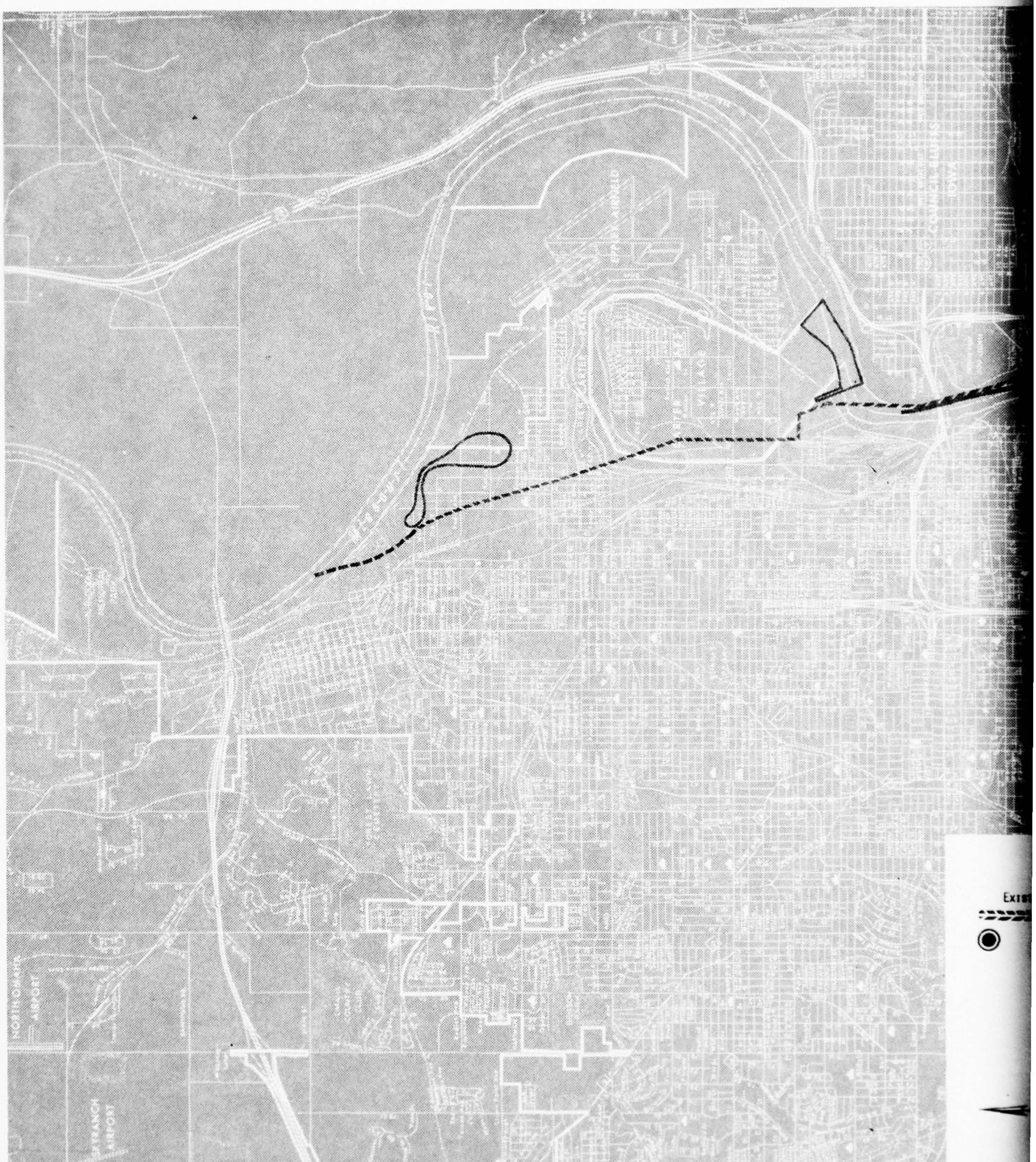


ALTERNATIVE NO. 5A
DEEP TUNNEL WITH MINED STORAGE
ALTERNATIVE PLANS FOR ABATEMENT OF
POLLUTION FROM COMBINED SEWER OVERFLOWS
OMAHA, NEBRASKA
PLATE 21

2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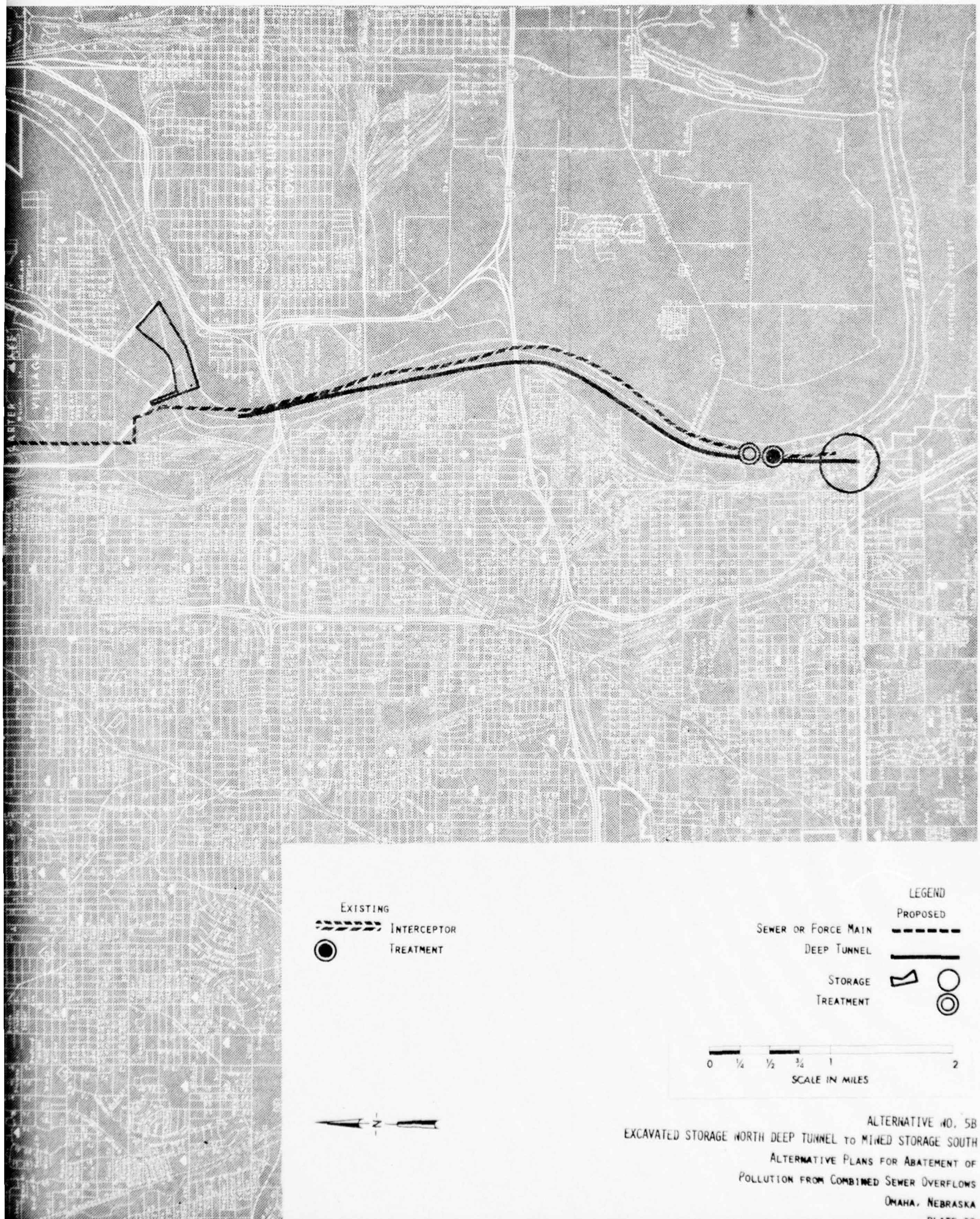
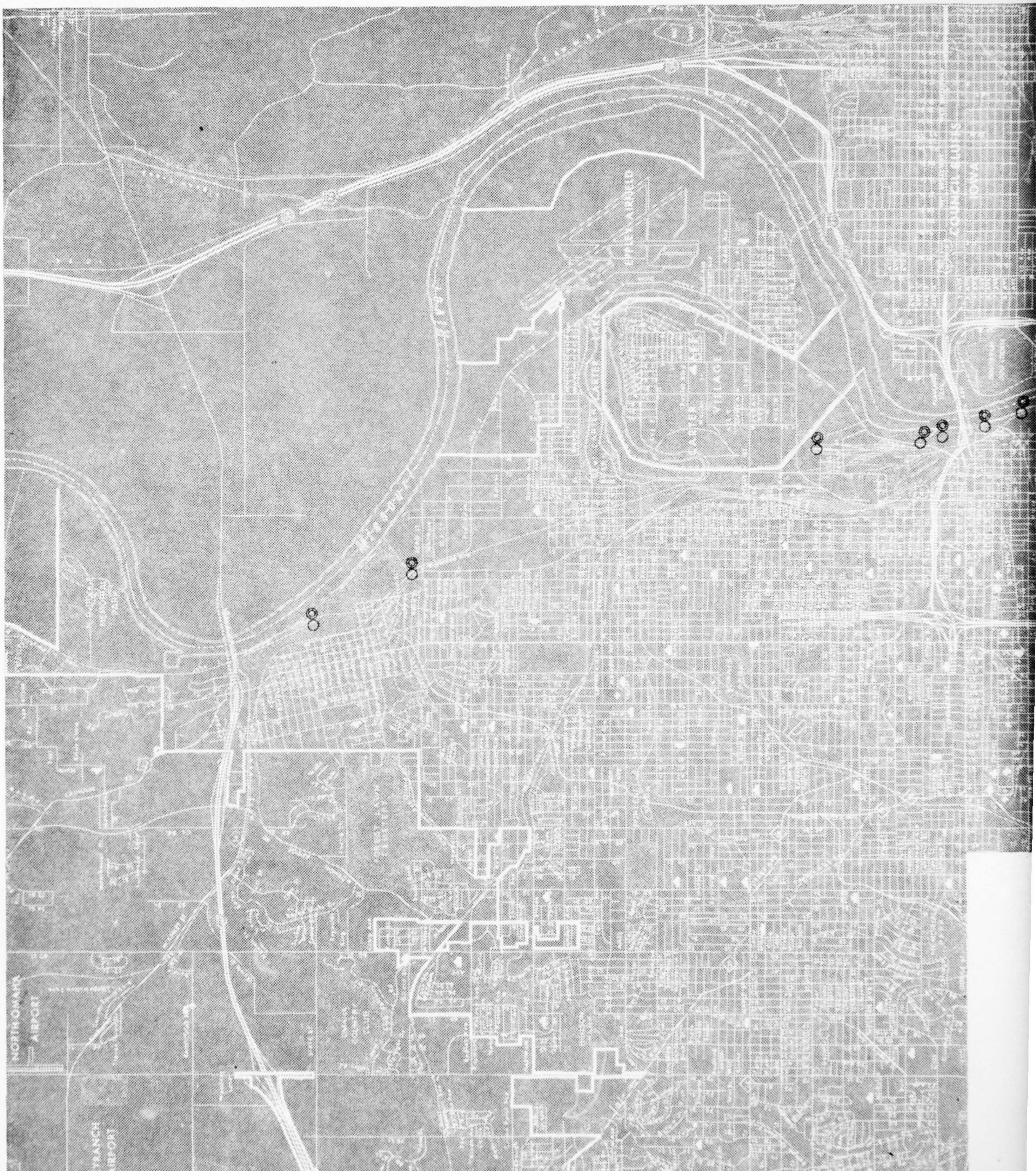
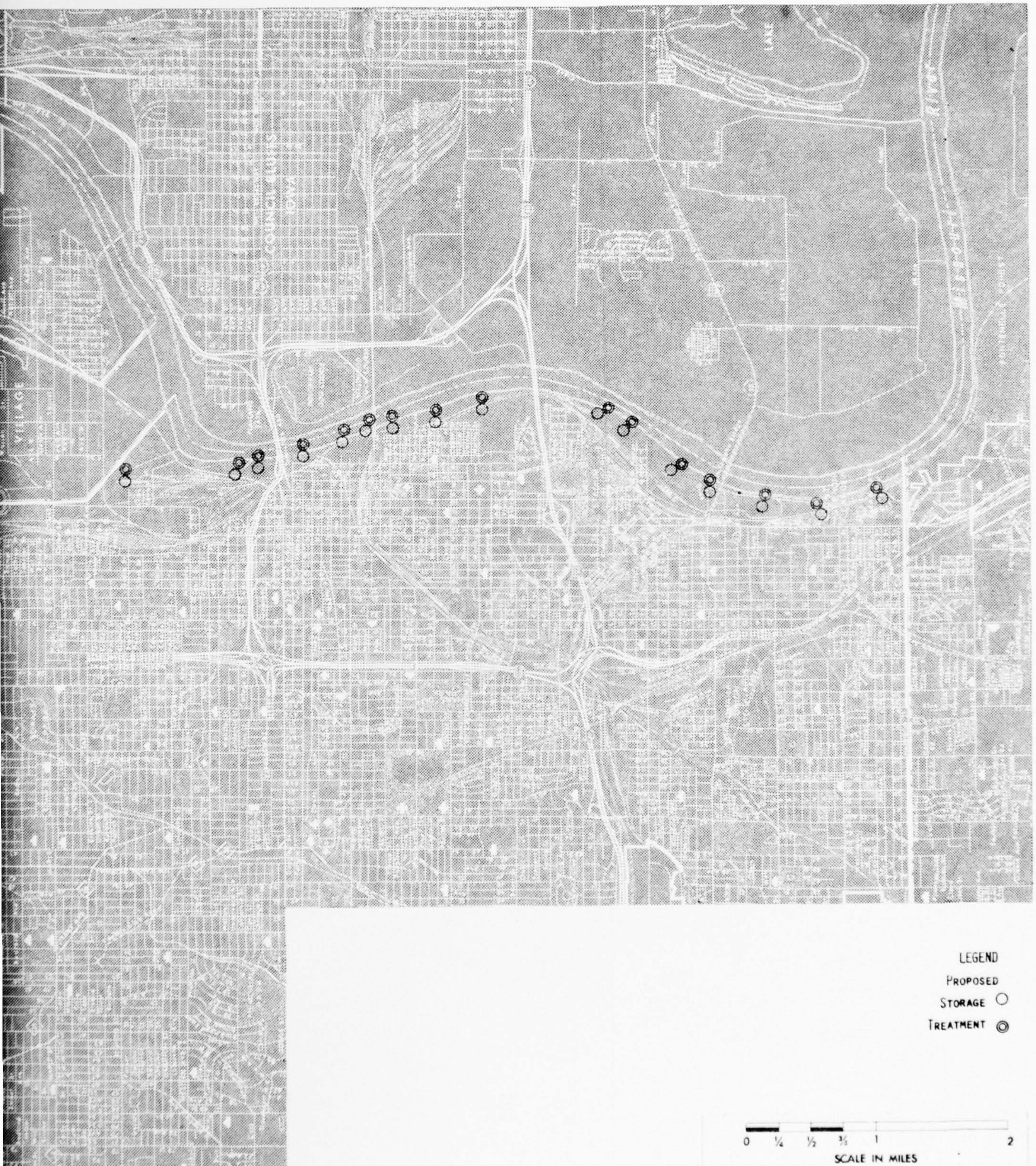


ALTERNATIVE NO. 5B
EXCAVATED STORAGE NORTH DEEP TUNNEL to MINED STORAGE SOUTH
Alternative Plans for Abatement of
Pollution from Combined Sewer Overflows
Omaha, Nebraska
PLATE 22

2





LEGEND

PROPOSED

STORAGE ○

TREATMENT ◎

0 ¼ ½ ¾ 1 2

SCALE IN MILES

N

ALTERNATIVE NO. 6

FLOW-THROUGH TREATMENT WITH STORAGE AT OUTFALLS

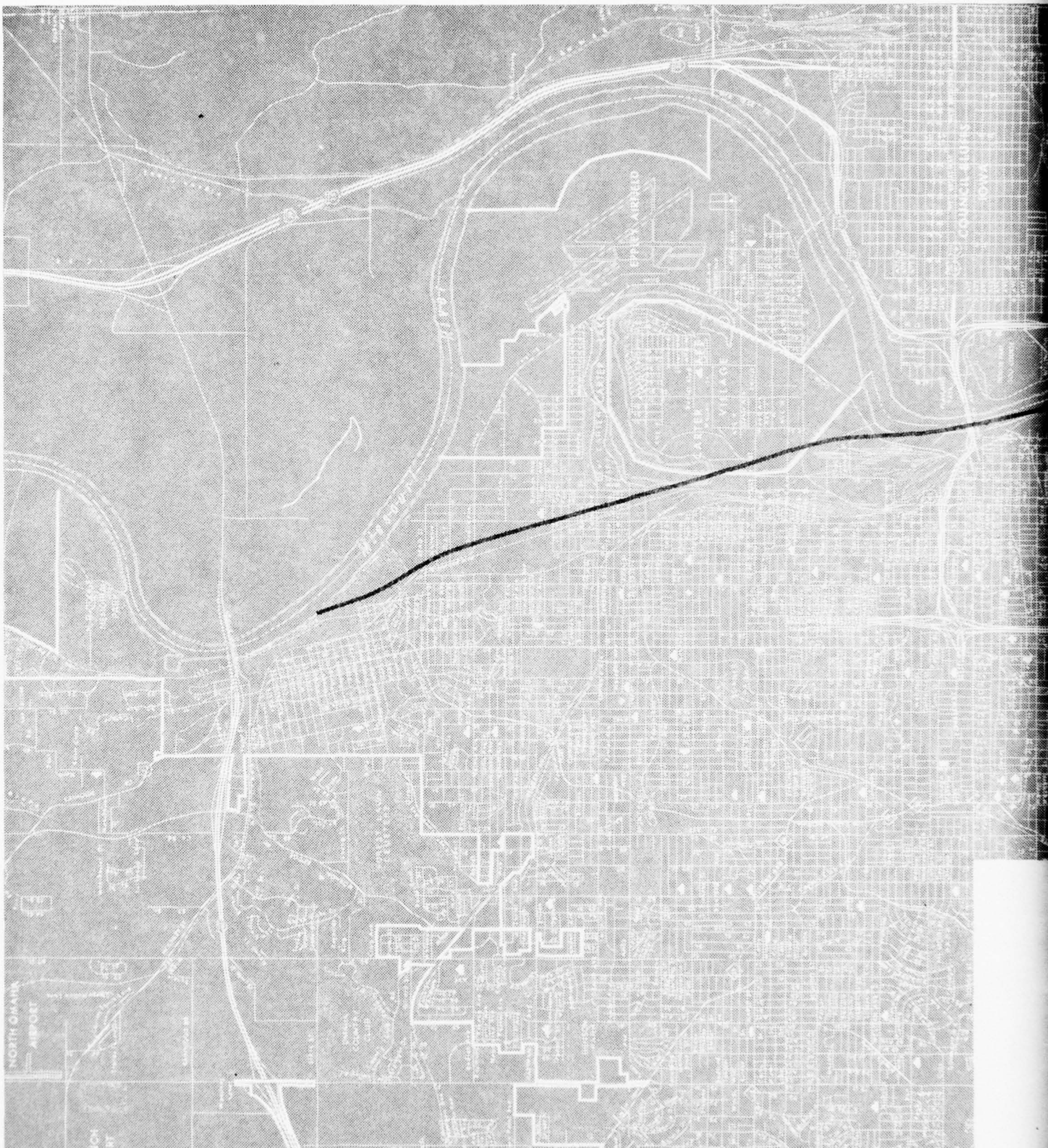
ALTERNATIVE PLANS FOR ABATEMENT OF

POLLUTION FROM COMBINED SEWER OVERFLOWS

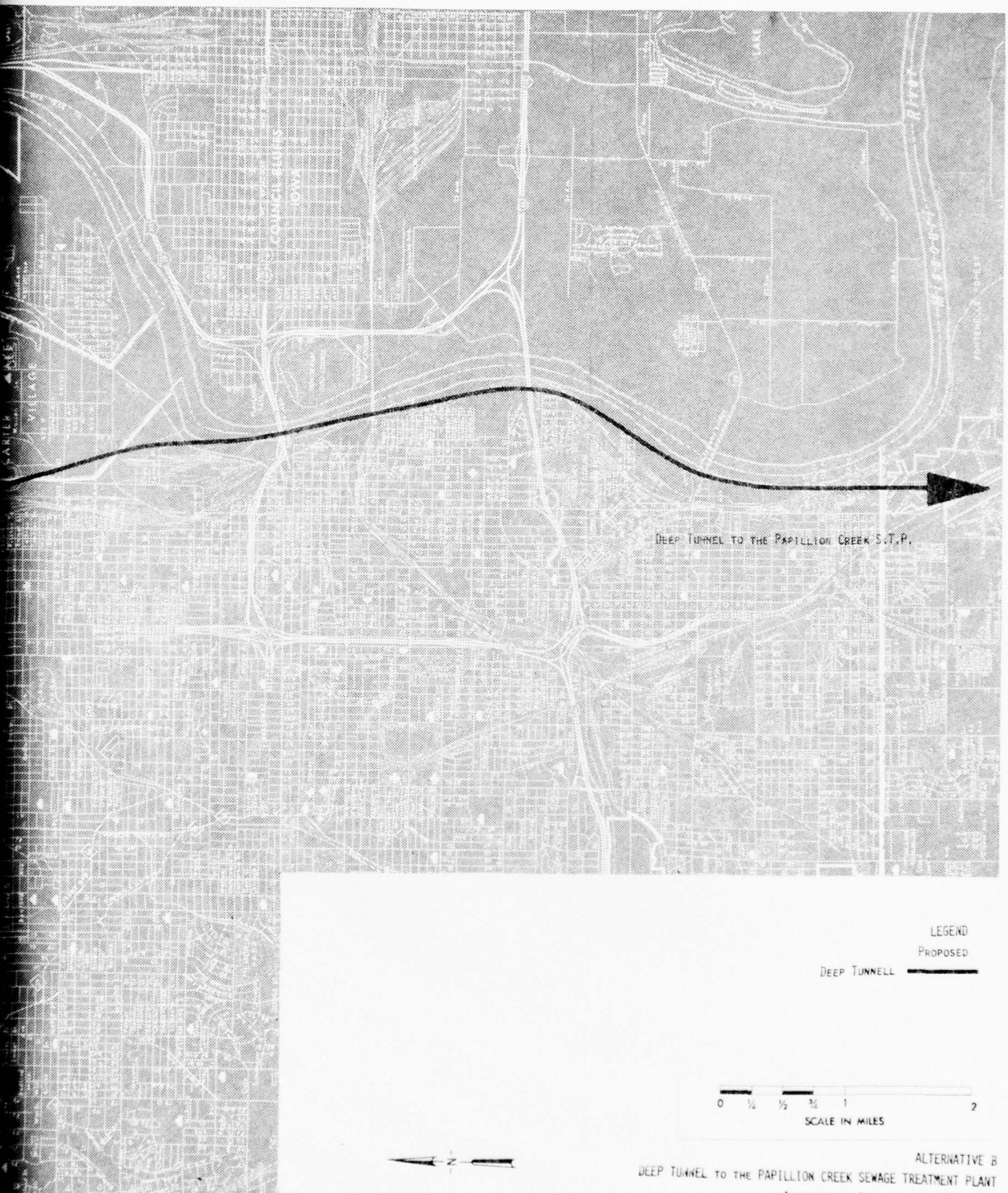
OMAHA, NEBRASKA

PLATE 23

2



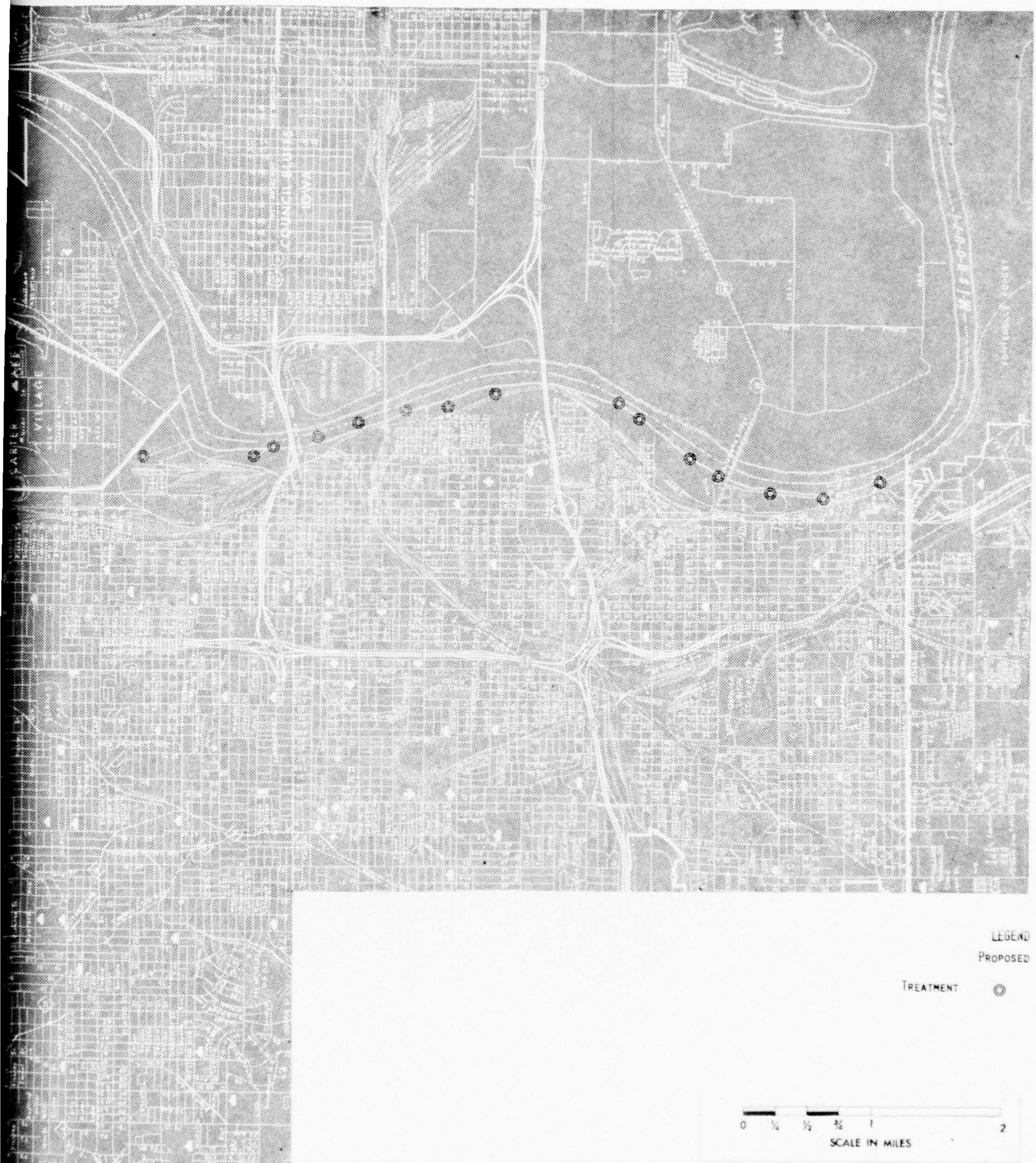
NORTH OMAHA AIRPORT
EPPLEY AIRFIELD
COUNCIL BLUFFS IOWA



ALTERNATIVE B
DEEP TUNNEL TO THE PAPILLION CREEK SEWAGE TREATMENT PLANT
Alternative Plans for Abatement of
Pollution from Combined Sewer Overflows
Omaha, Nebraska
PLATE 24

2


EPPLEY AIRFIELD
COUNCIL BLUFFS
IOWA
NORTH OMAHA
AIRPORT



ALTERNATIVE C

FLOW-THROUGH TREATMENT FOR "FIRST-FLUSH"

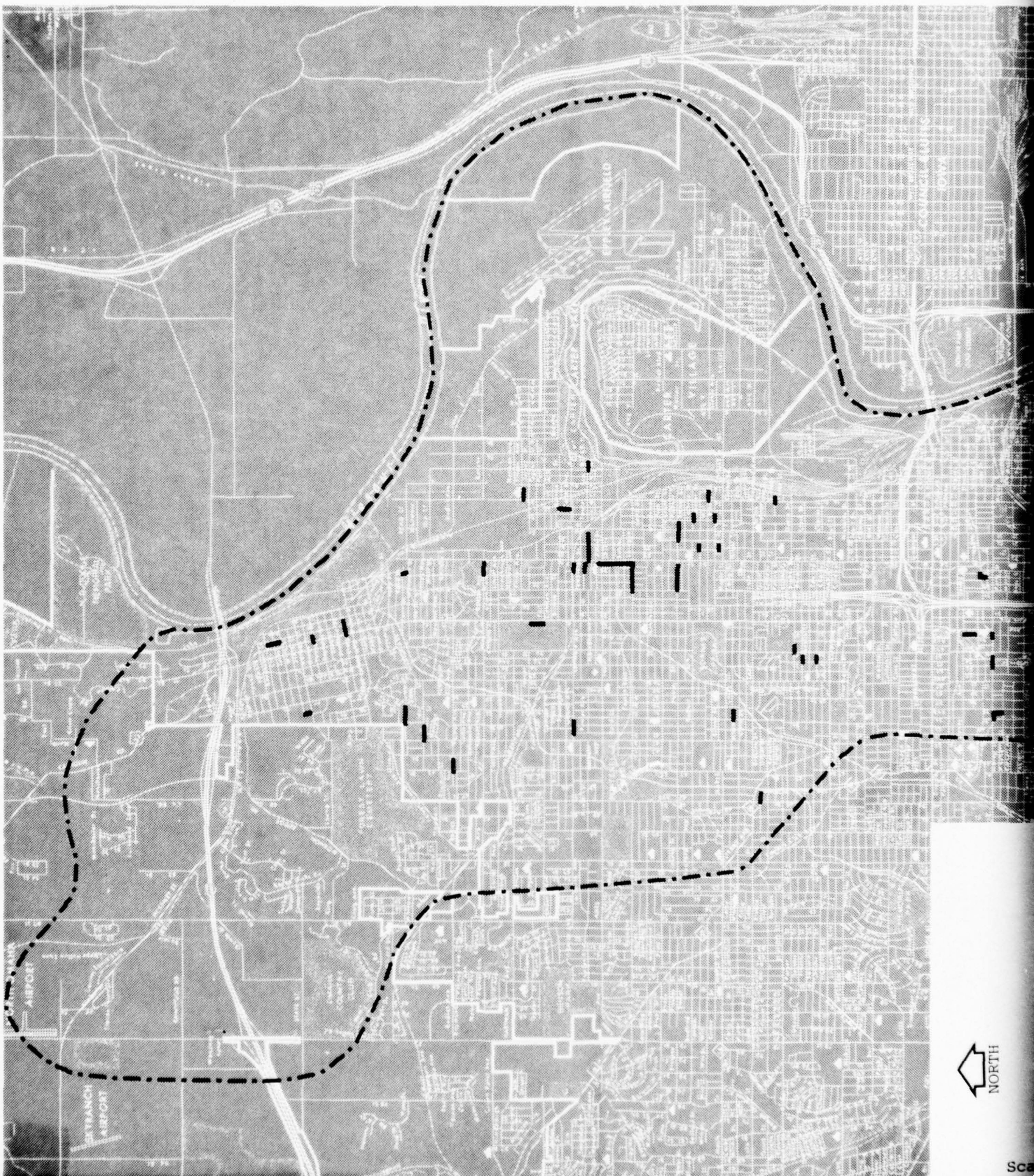
ALTERNATIVE PLANS FOR ABATEMENT OF

POLLUTION FROM COMBINED SEWER OVERFLOWS

OMAHA, NEBRASKA

PLATE 25

2



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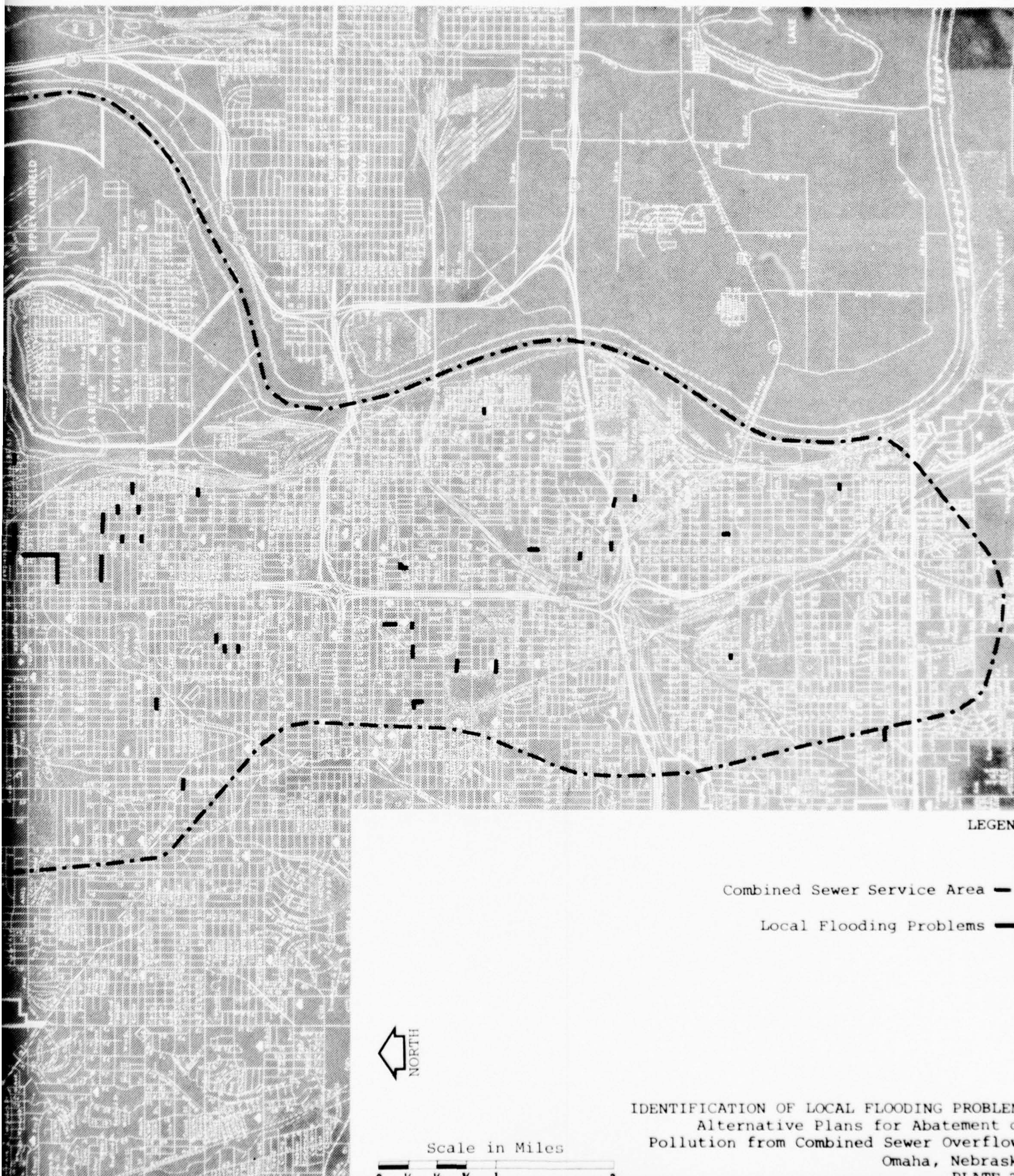


LEGEND

Combined Sewer Service Area

Local Flooding Problems

NORTH

Scale in Miles

0 ¼ ½ ¾ 1 2

IDENTIFICATION OF LOCAL FLOODING PROBLEMS
Alternative Plans for Abatement of
Pollution from Combined Sewer Overflows
Omaha, Nebraska
PLATE 26

COST ESTIMATES OF ALTERNATIVE PLANS

FOR 5 YEAR RECURRENCE INTERVAL

AND LEVEL 1 TREATMENT

(Costs in Millions of Dollars)

	Alternative Plan	Construction Cost							
		Conveyance Piping	Drop Shafts	Tunnels	Storage Structures	Aeration	Pump Stations & App.1/	Treat-ment	Land
2	Diked Storage along Levee	20.0	-	-	32.0	7.8	4.2	7.7	5.0
4A	Deep Tunnel to Ground Level Storage	1.5	15.4	91.0	6.8	7.8	14.4	7.7	2.5
4B	Excavated Storage - Deep Tunnel	1.9	15.4	38.3	31.7	7.8	8.2	7.7	3.0
5A	Deep Tunnel w/Mined Storage	-	21.0	49.9	66.9	15.6	17.7	7.7	-
B	Deep Tunnel to Papillion Creek	-	21.0	63.6	66.9	15.6	22.2	2/	-

	Alternative Plan	Operation & Maintenance				Totals		Present Worth	
		Conveyance & Tunnel	Pumping Stations	Aeration & Sludge Handling	Treatment	Total Annual O&M	Total Constr	Present Worth O&M3/	Total Present Worth4/
2	Diked Storage along Levee	-	0.1	3.9	0.7	4.7	71.1	67.9	143.9
4A	Deep Tunnel to Ground Level Storage	0.1	0.4	3.9	0.7	5.1	144.6	74.8	221.9
4B	Excavated Storage - Deep Tunnel	0.1	0.4	3.9	0.7	5.1	111.0	72.6	186.6
5A	Deep Tunnel w/Mined Storage	0.2	0.3	4.9	0.7	6.1	178.8	91.3	270.1
B	Deep Tunnel to Papillion Creek	0.4	0.3	4.9	2/	5.6	189.5	86.6	276.1

1/ Includes preliminary treatment, such as, grit and sludges removal facilities.

2/ Does not include treatment-regional concept.

3/ Includes costs for replacement.

4/ Includes costs for land.

PRESENT WORTH COSTS FOR

ALTERNATIVE PLANS

(Costs in Million of Dollars)

Alternative		Treatment Level	Recurrence Interval - Years			
			1	2	5	10
2	Diked Storage Along Levee	1	98.9	115.7	143.9	157.3
		2	133.9	155.7	193.9	217.3
		3	163.9	200.7	253.9	282.3
4a	Deep Tunnel to Ground Level Storage	1	149.7	171.8	221.9	237.3
		2	184.7	211.8	271.9	297.3
		3	224.7	256.8	331.9	362.3
4b	Excavated Storage No. Deep Tunnel So.	1	133.5	152.4	186.6	202.2
		2	168.5	192.4	236.5	262.2
		3	198.5	237.4	296.5	327.2
5a	Deep Tunnel w/ Mined Storage	1	189.6	222.1	270.1	299.0
		2	224.6	262.1	320.1	359.0
		3	254.6	307.1	380.1	424.3
B	Deep Tunnel to Papillion Creek1/	2/	196.6	227.4	276.1	302.5

1/ Includes conveyance of primary effluent from Missouri River Treatment Plant.

2/ Treatment not included.

SUMMARY OF ENVIRONMENTAL ASSESSMENT FACTORS

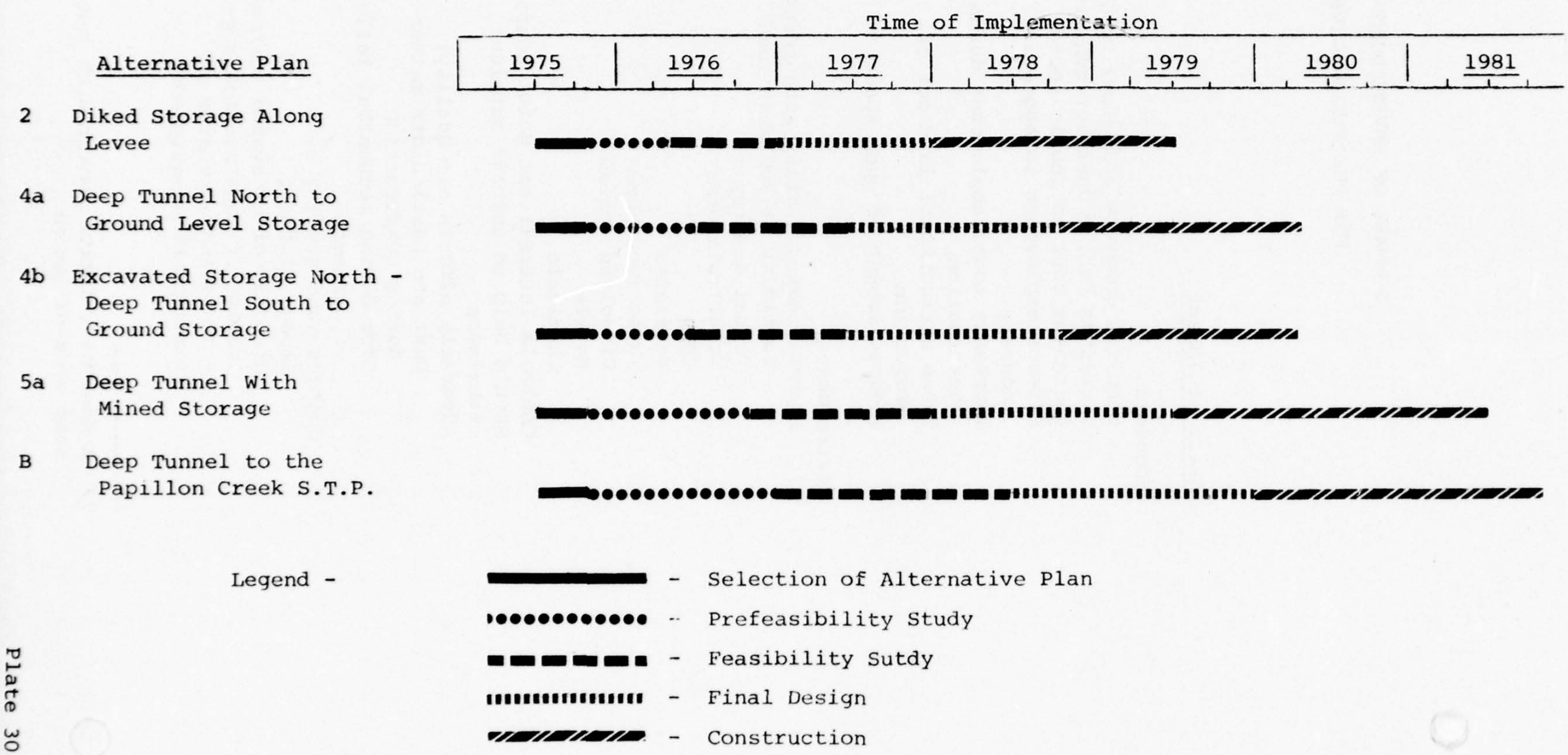
FOR THE ALTERNATIVE PLANS STUDIES

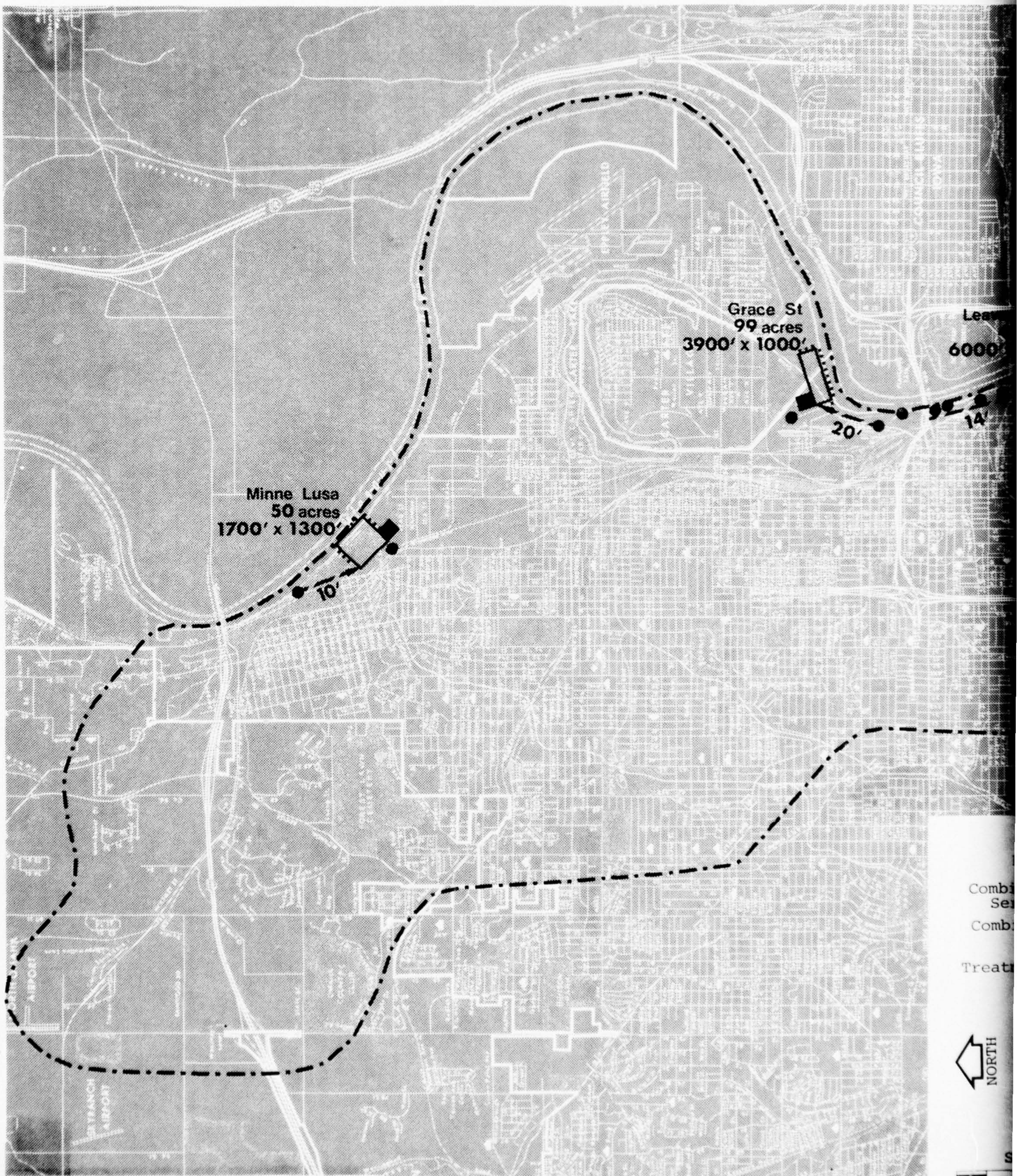
Areas of Impact	Alternative Plan Impact[1/] 2	4a	4b	5a	B
Economic					
Requires concrete and steel resources	x	xx	xx	xxx	xxx
Increases public contract construction	x	x	x	x	x
Increases cost to public sector	x	x	x	x	x
Affects employment in aggregate industry		x	x	xx	xx
Increases local employment during construction	xx	x	x	x	x
Takes agricultural land out of production		x	x		
Reduces amount of open space	x				
Environmental					
Improves water quality with respect to:					
Turbidity or sediment load	x	x	x	x	x
Visual quality	x	x	x	x	x
Fecal coliforms	x	x	x	x	x
BOD	x	x	x	x	x
Nutrients	x	x	x	x	x
Toxic substances	x	x	x	x	x
Dissolved solids	x	x	x	x	x
Metals	x	x	x	x	x
Chemicals	x	x	x	x	x
Controls intermittent point source	x	x	x	x	x
Should help to improve nutrient imbalance	x	x	x	x	x
Adversely affects air quality:					
Dust and particulate matter during construction		x	x	x	x
Odor during mechanical failure of aerators	x	x	x		
Changes land quality by:					
Sludge disposal	x	x	x	x	x
Disposal of excavated aggregate	xx				
Storage of construction aggregate		x	x	x	x
Storage and servicing of construction equipment	xx	x	x	x	x

1/ X denotes relative impact with respect to other alternatives for same area of impact.

SUMMARY OF IMPLEMENTATION SCHEDULE

OF ALTERNATIVE PLANS





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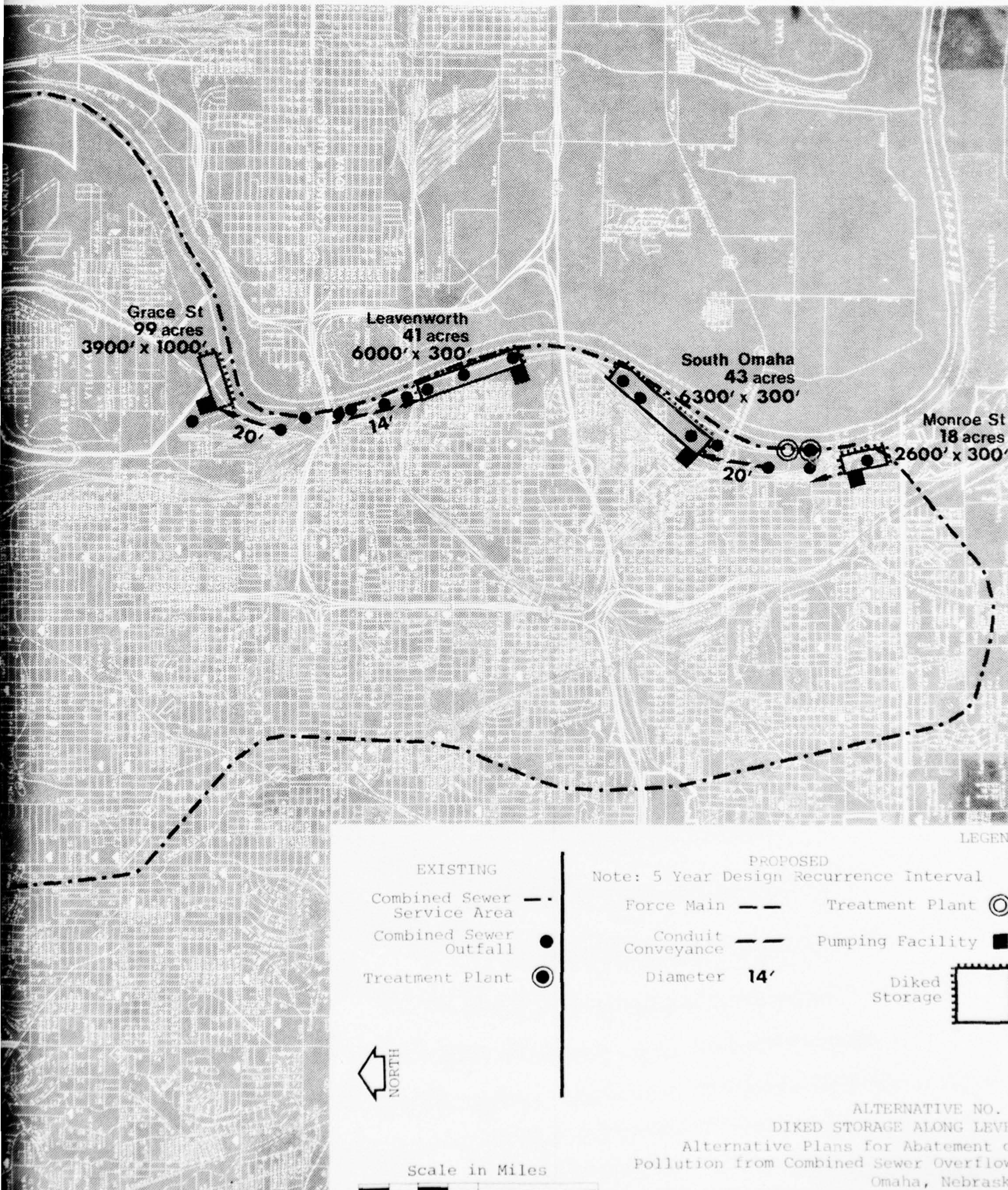

ALTERNATIVE NO. 2
DIKED STORAGE ALONG LEVEE
Alternative Plans for Abatement of
Pollution from Combined Sewer Overflows
Omaha, Nebraska
PLATE 31

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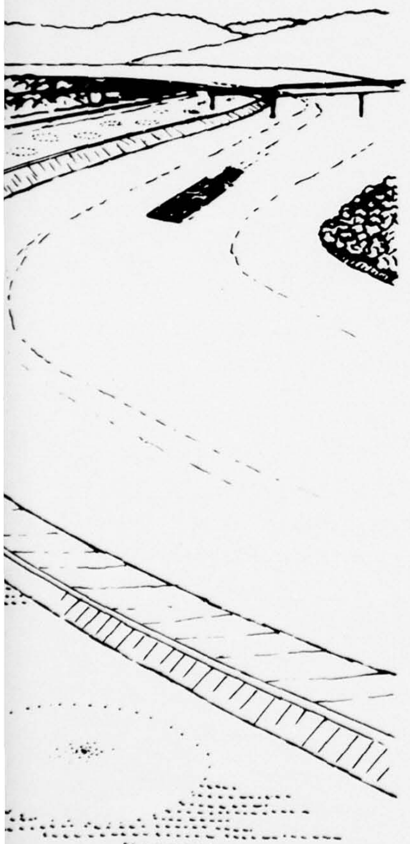
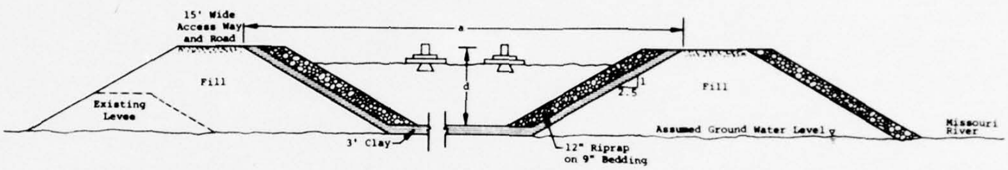




TYPICAL CROSS SECTION
AERATED RESERVOIR CHANNEL WITH DIKED STORAGE ALONG RIVERFRONT

For Dimensional Data

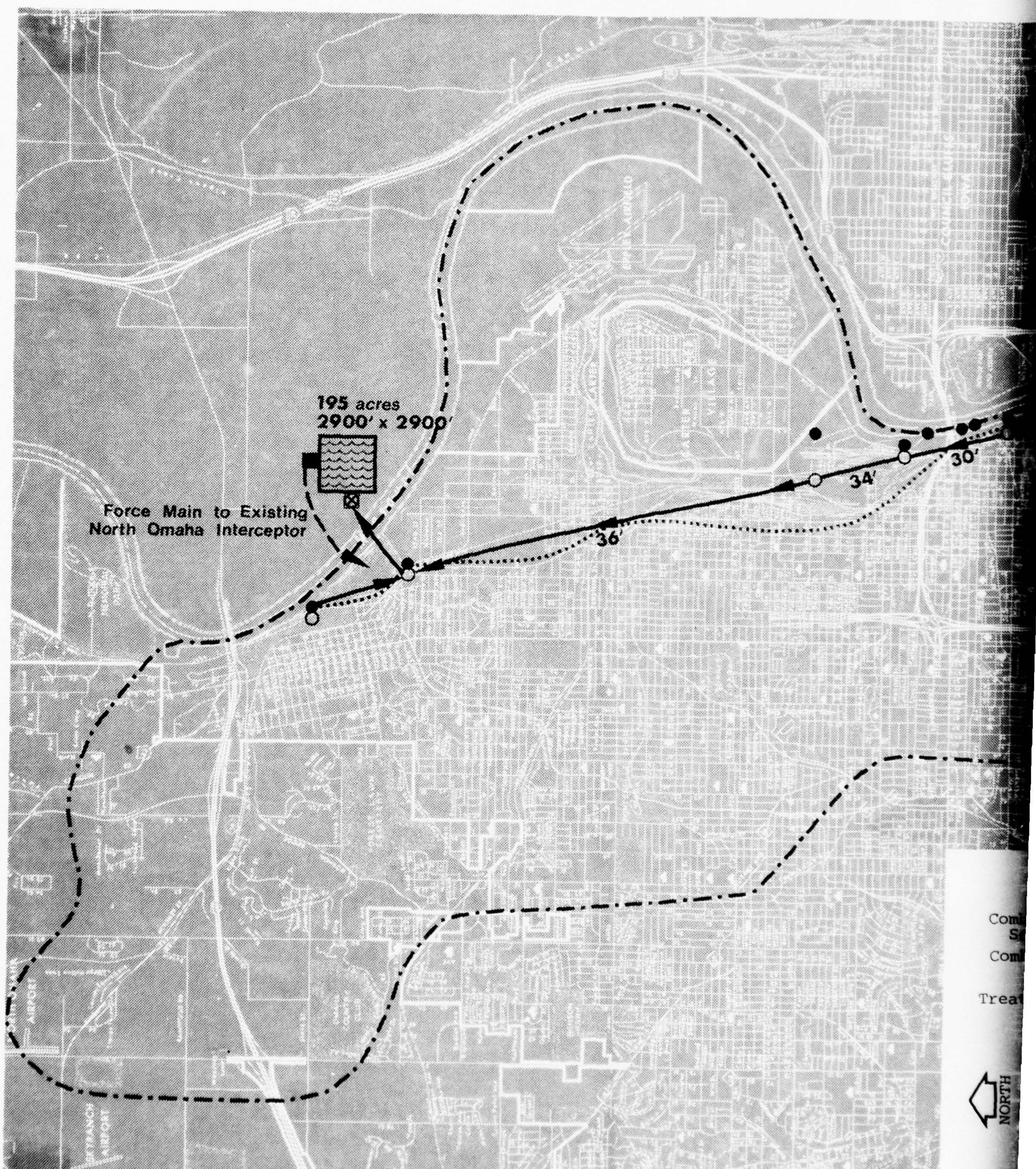
See Table VII - 2.

DIKED STORAGE RESERVOIR

Alternative Plans for Abatement of
Pollution from Combined Sewer Overflows
Omaha, Nebraska

PLATE 32

2



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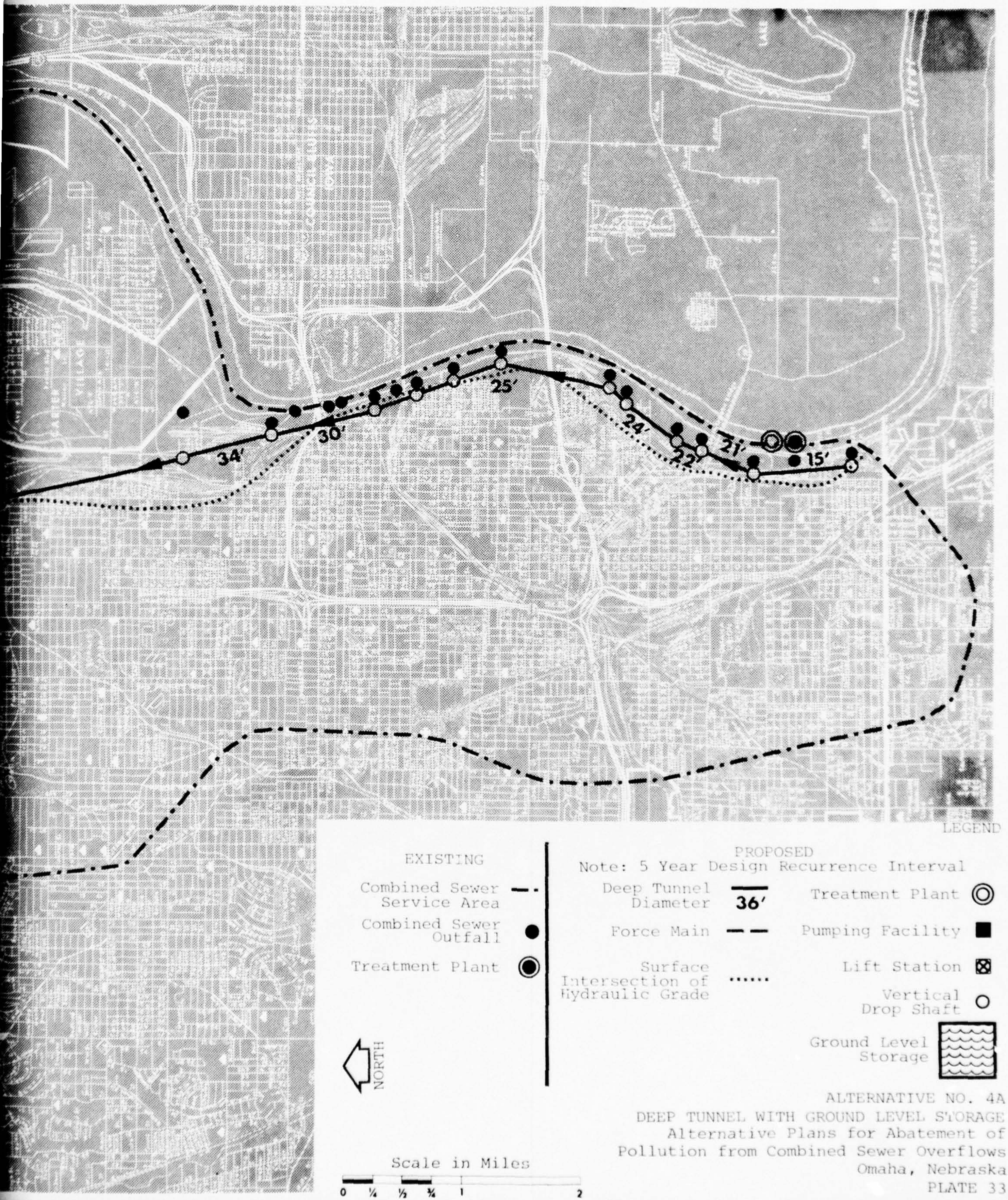
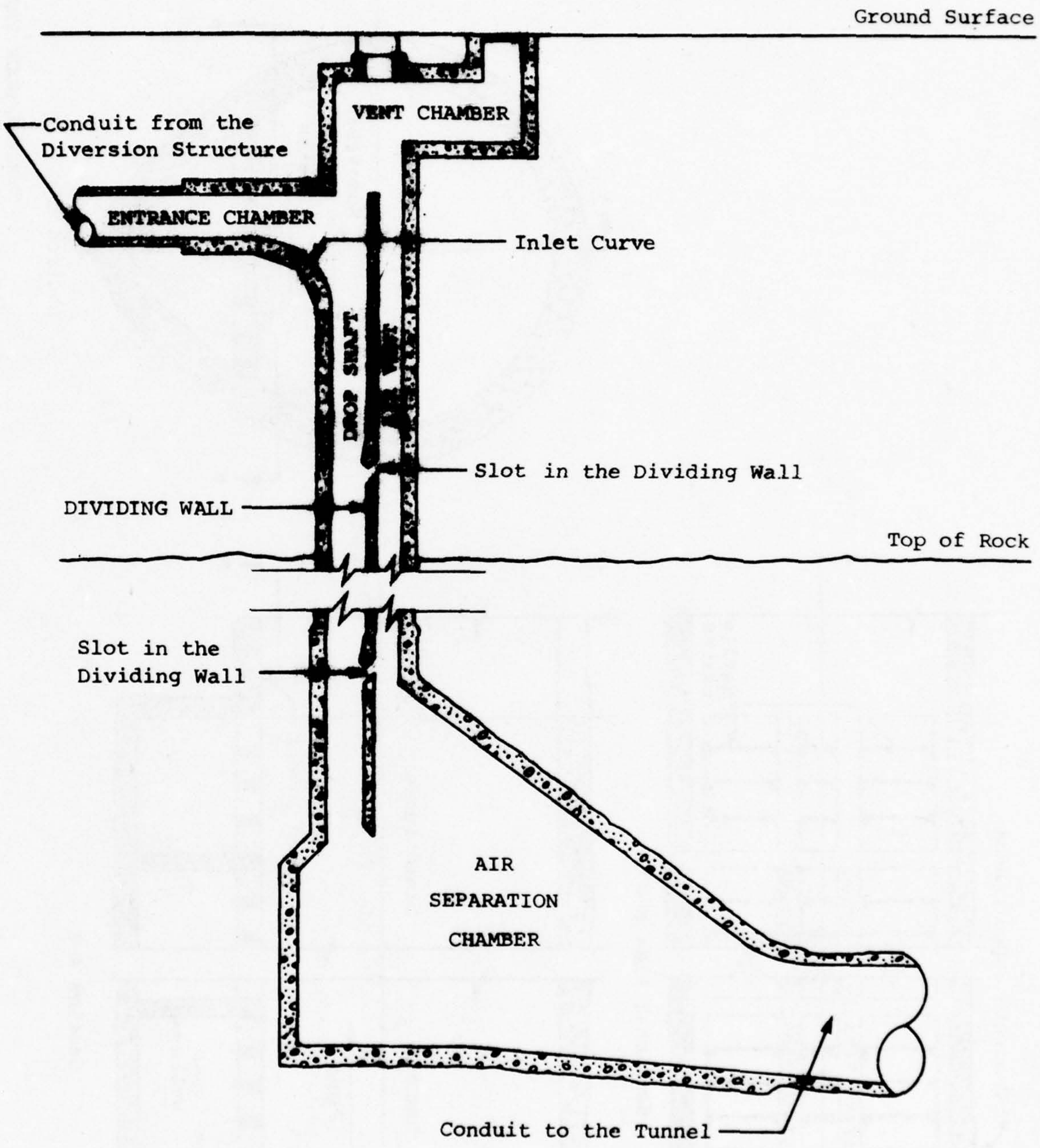


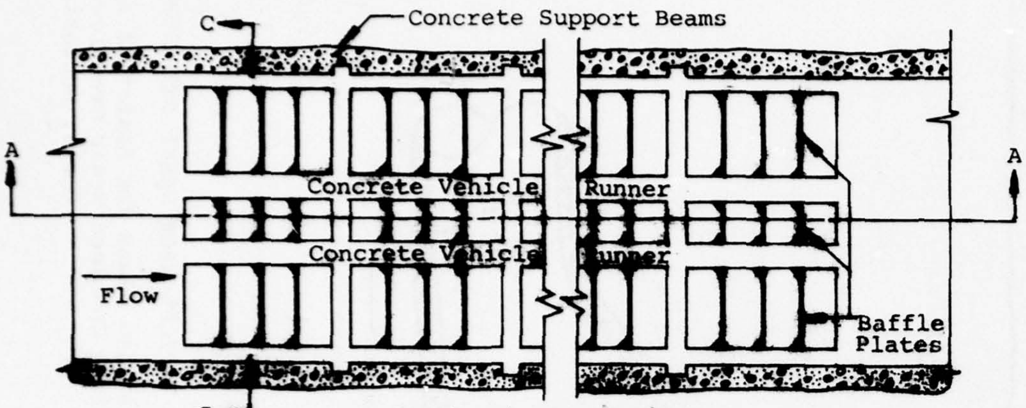
ALTERNATIVE NO. 4A
DEEP TUNNEL WITH GROUND LEVEL STORAGE
Alternative Plans for Abatement of
Pollution from Combined Sewer Overflows
Omaha, Nebraska
PLATE 33

2

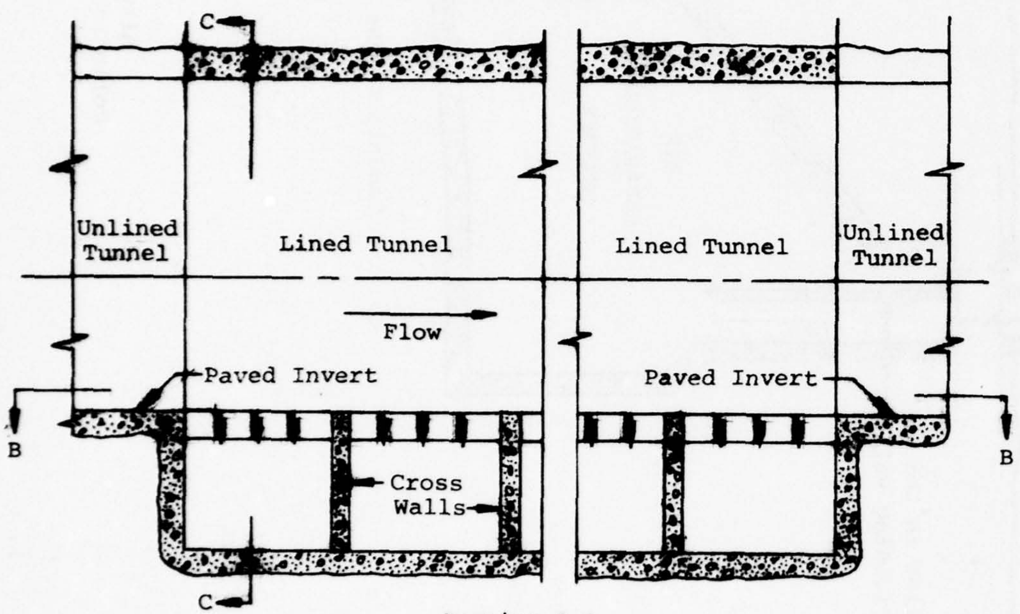


TYPICAL DROP SHAFT STRUCTURE

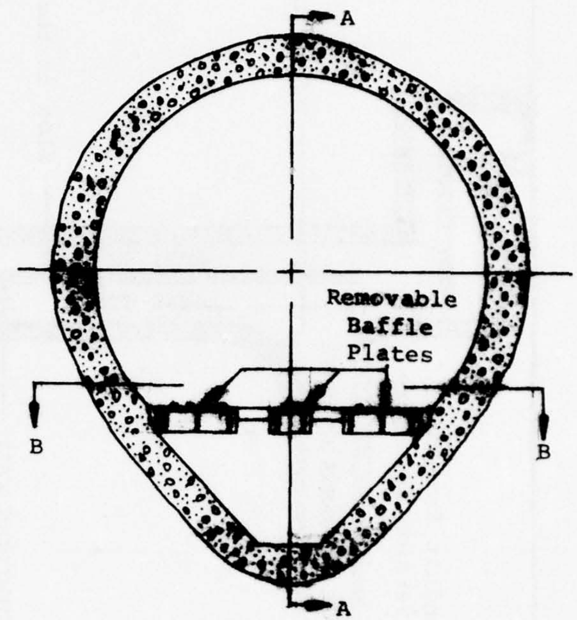
Alternative Plans for Abatement of
Pollution from Combined Sewer Overflows
Omaha, Nebraska



Sectional Plan B-B



Section A-A



Section C-C

TYPICAL ROCK TRAP

Alternative Plans for Abatement of
Pollution from Combined Sewer Overflows
Omaha, Nebraska

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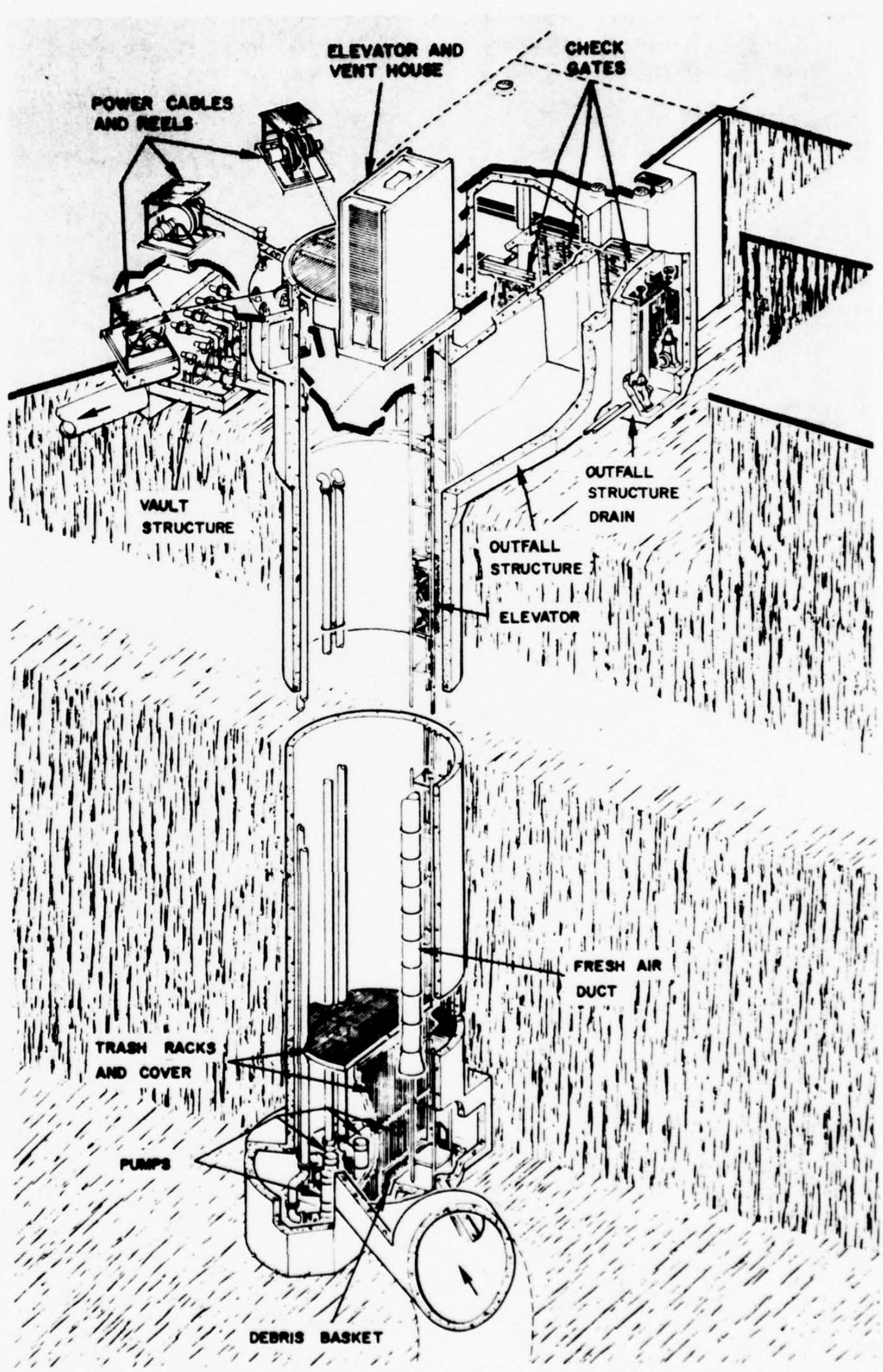
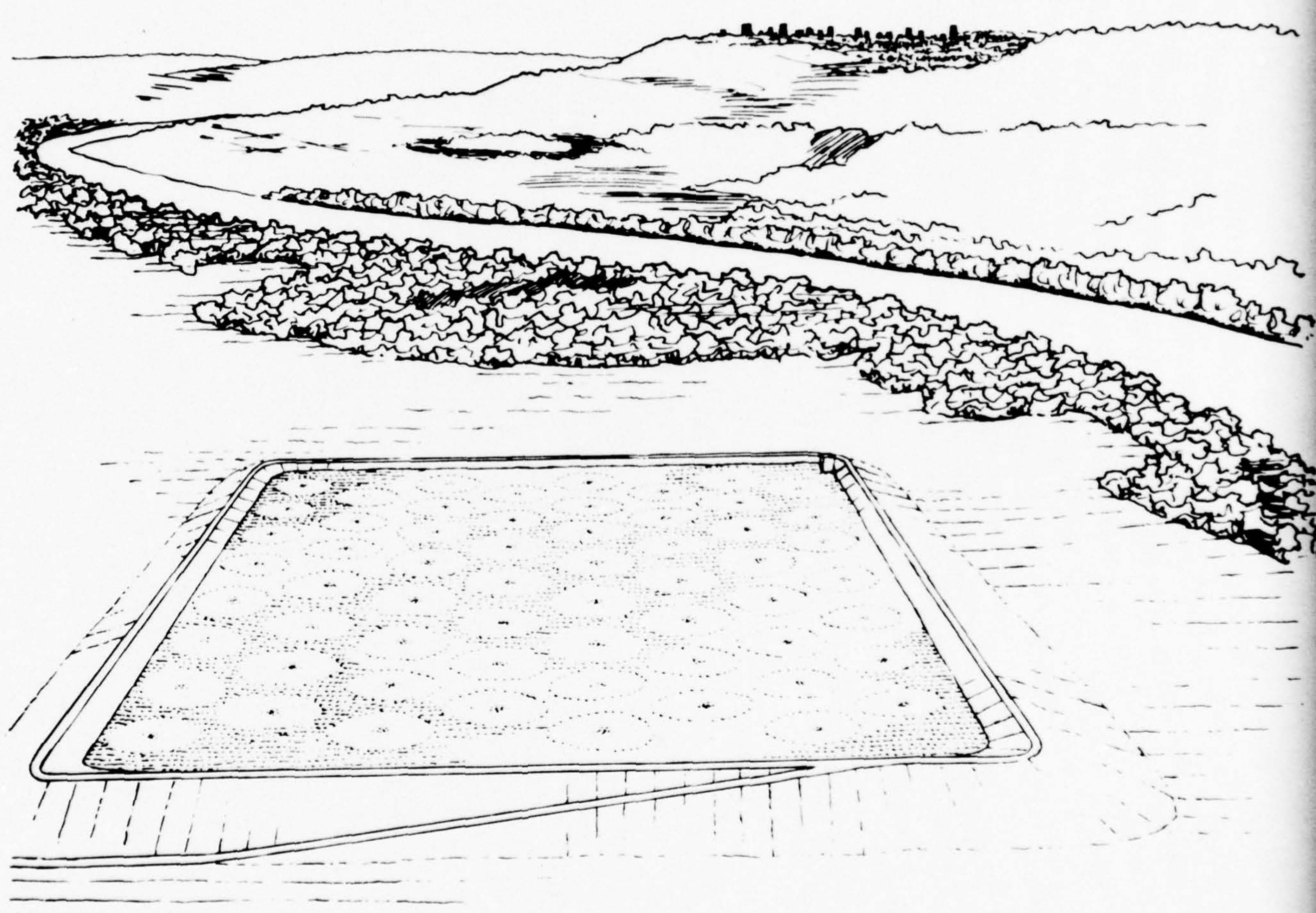


TYPICAL LIFT STATION

Alternate Plans for Abatement of
Pollution from Combined Sewer Overflows
Omaha, Nebraska



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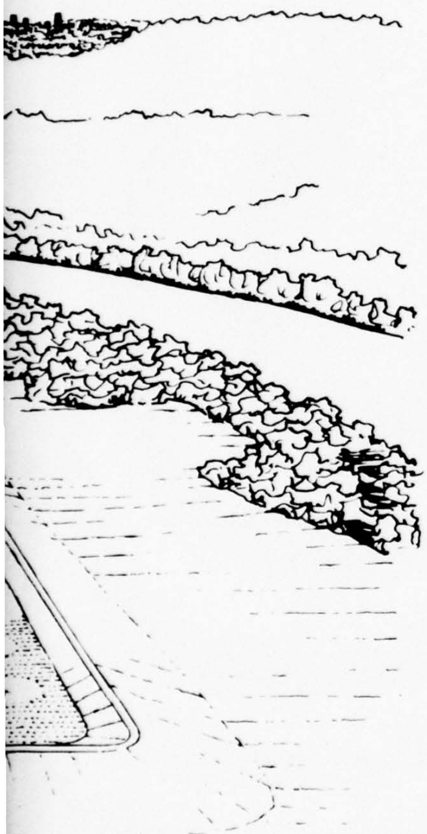
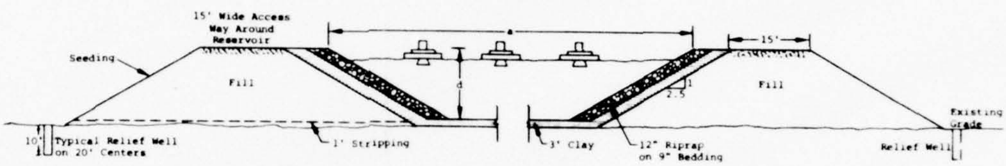




TYPICAL CROSS SECTION
AERATED GROUND LEVEL STORAGE

DIMENSIONAL DATA

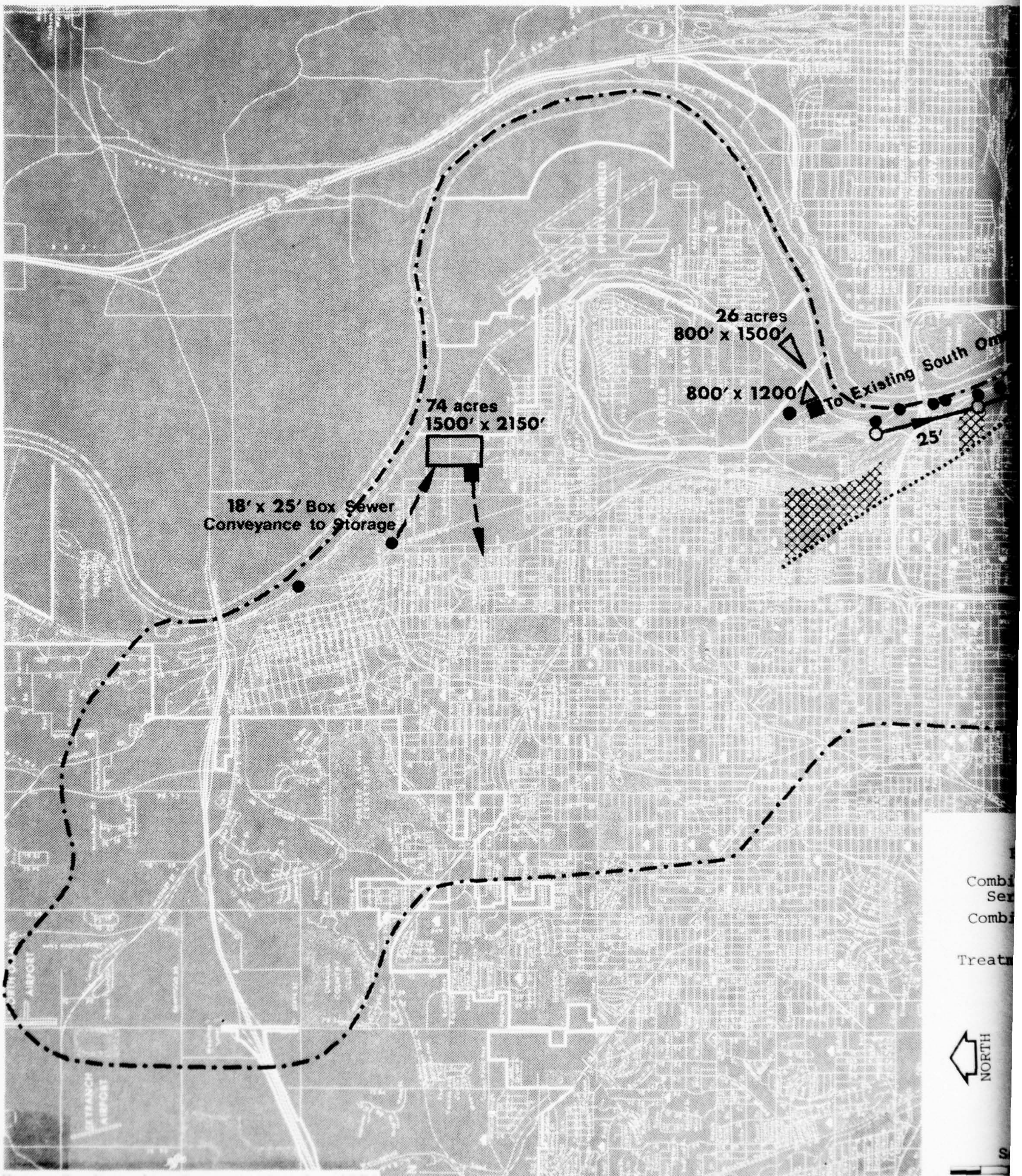
Recurrence Interval (years)	Alternative 4A			Alternative 4B		
	a* (feet)	d (feet)	surface acres	a (feet)	d (feet)	surface acres
1	2300	15	120	1800	15	75
2	2550	15	150	2050	15	100
5	2950	15	200	2350	15	130
10	3200	15	230	2550	15	150

* assuming square reservoir

TYPICAL GROUND LEVEL STORAGE RESERVOIR

Alternative Plans for Abatement of
Pollution from Combined Sewer Overflows
Omaha, Nebraska

PLATE 37



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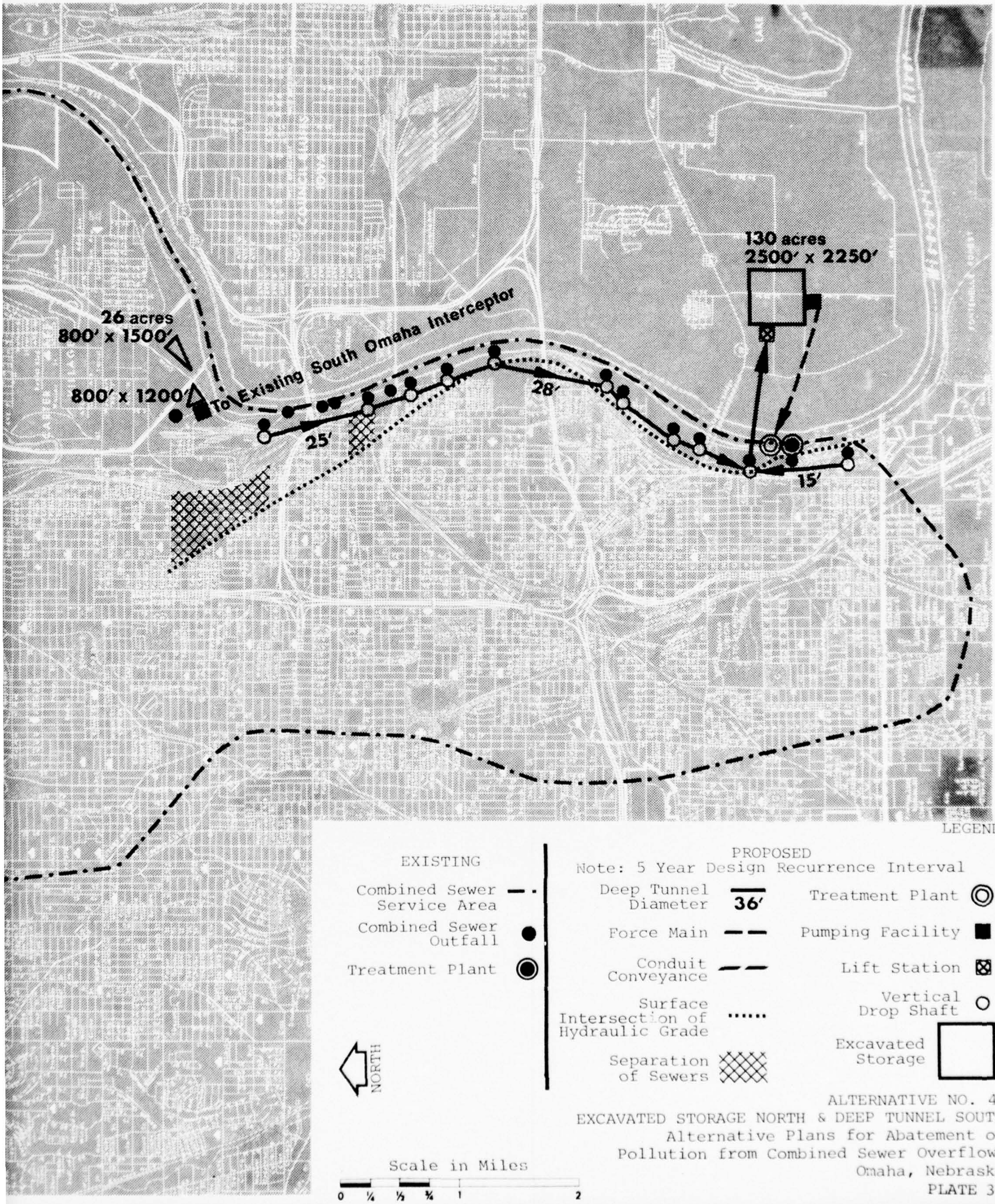
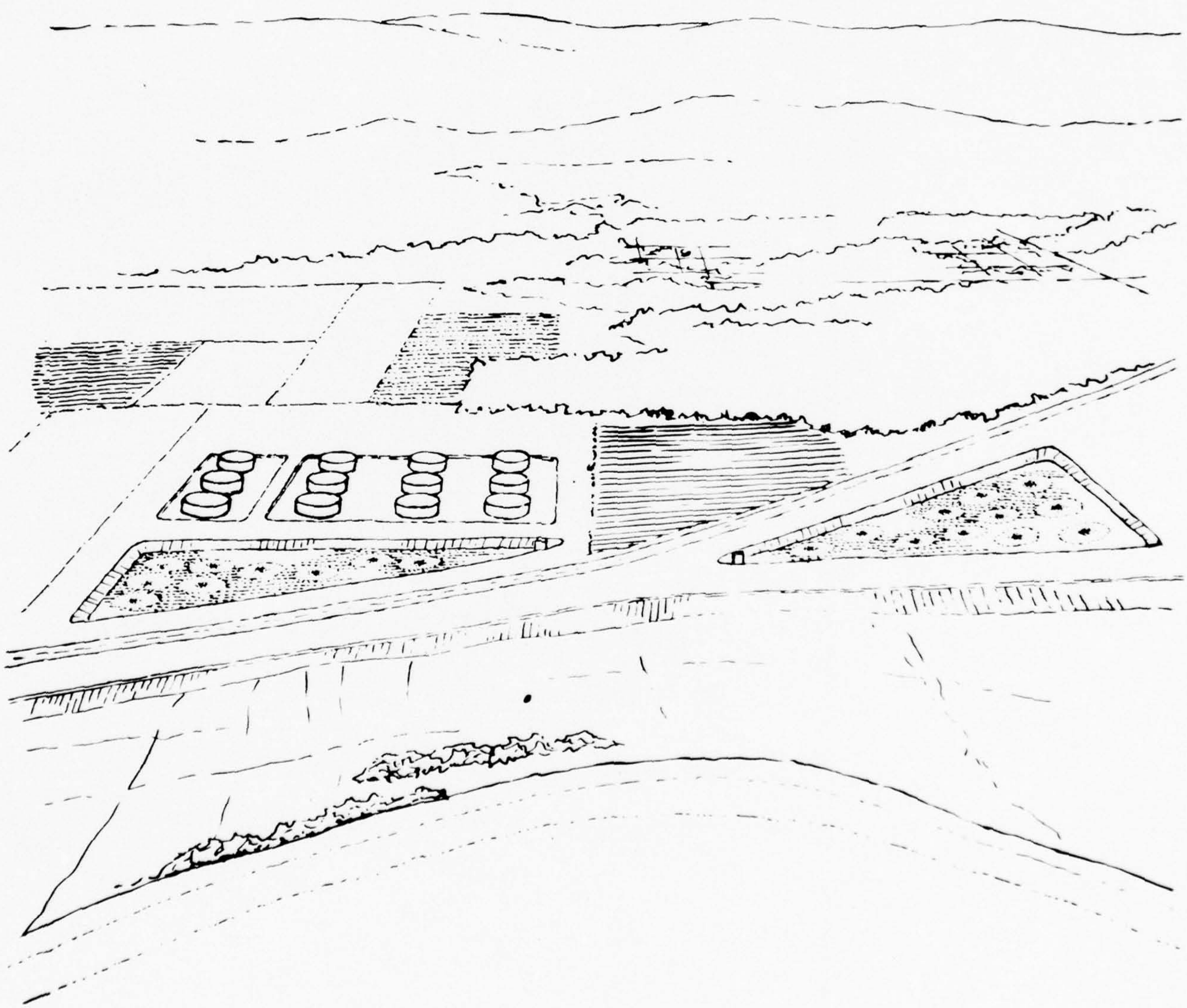


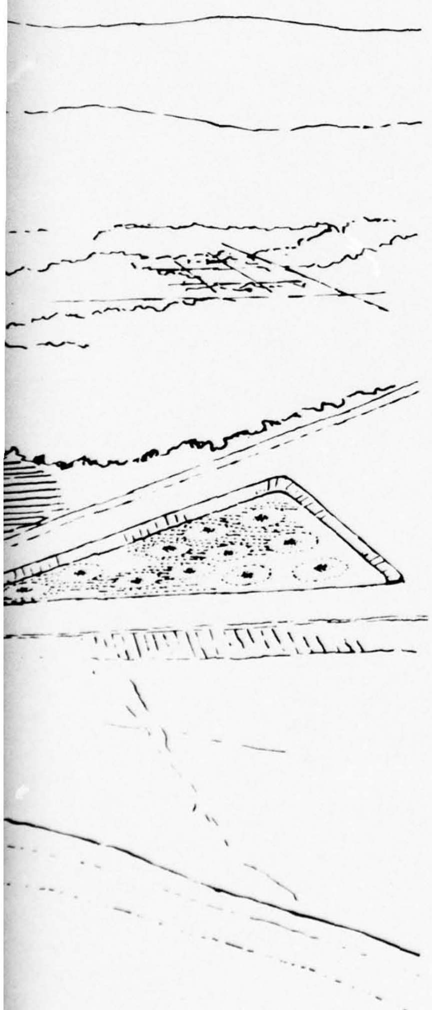
ALTERNATIVE NO. 4B
EXCAVATED STORAGE NORTH & DEEP TUNNEL SOUTH
Alternative Plans for Abatement of
Pollution from Combined Sewer Overflows
Omaha, Nebraska
PLATE 38

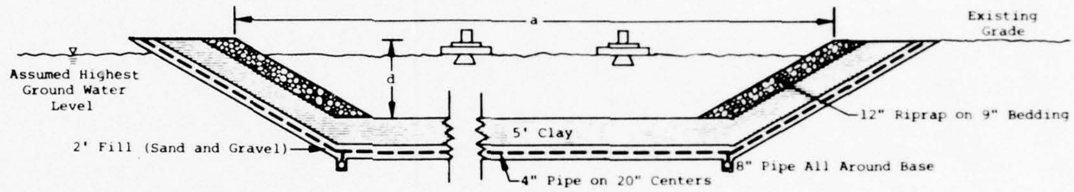
2



Assumed
Ground
Level

2'





TYPICAL CROSS SECTION
AERATED EXCAVATED STORAGE
(Carter Lake Area)

DIMENSIONAL DATA

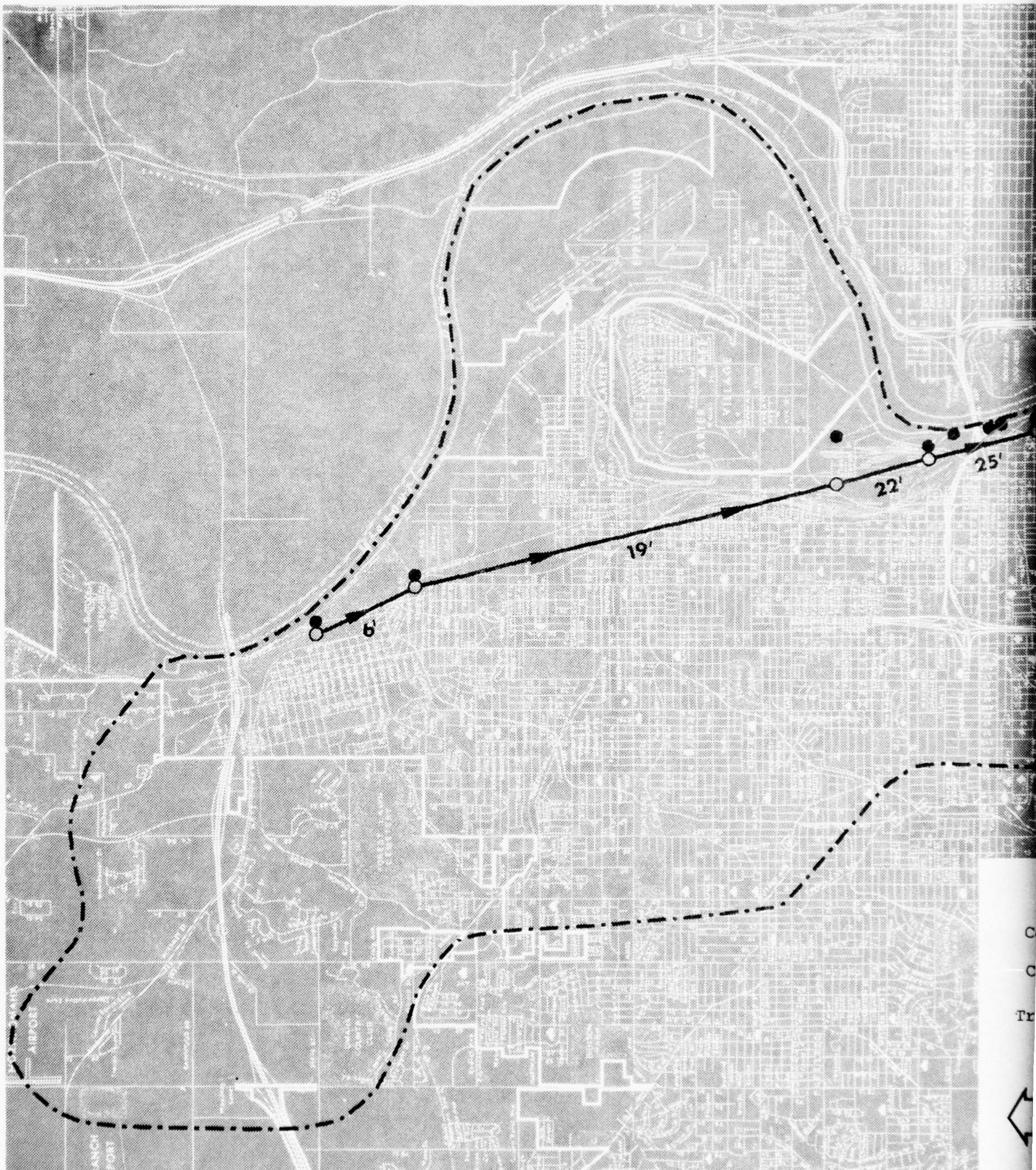
Recurrence Interval (years)	Minne Lusa Reservoir Alternative 4B			Grace St Reservoir North			South	
	a 1/ (feet)	d (feet)	surface acres	a x b 2/ (feet)	surface acres	d (feet)	a x b (feet)	surface acres
1	1500	15	50	800 x 1200	11	15	700 x 700	6
2	1550	15	55	800 x 1200	11	15	800 x 1000	9
5	1800	15	75	800 x 1200	11	15	800 x 1500	14
10	2000	15	90	800 x 1200	11	15	800 x 2000	19

1/ use square reservoir
2/ use triangular reservoir

TYPICAL EXCAVATED STORAGE RESERVOIRS

Alternative Plans for Abatement of
Pollution from Combined Sewer Overflows
Omaha, Nebraska

PLATE 39



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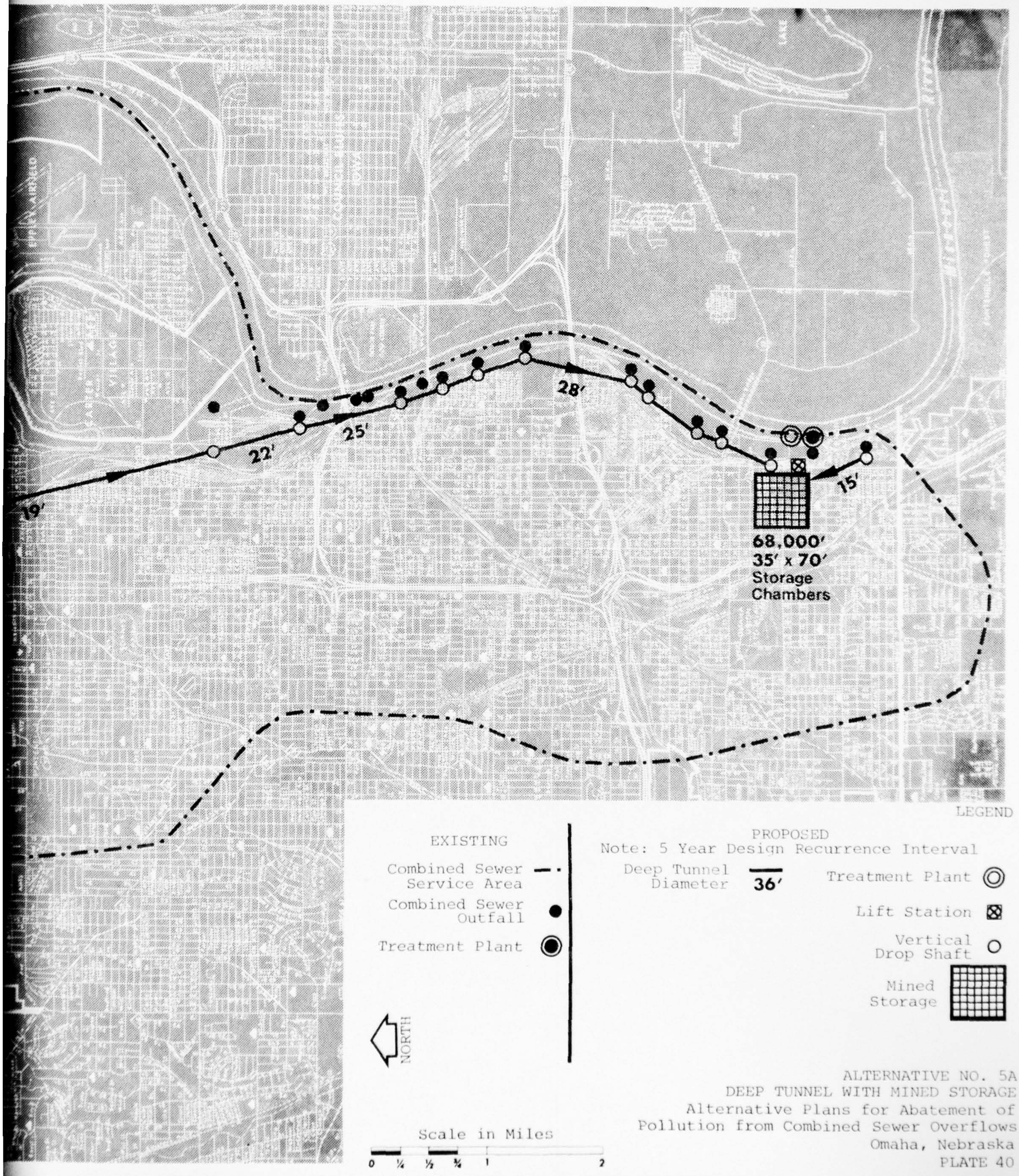
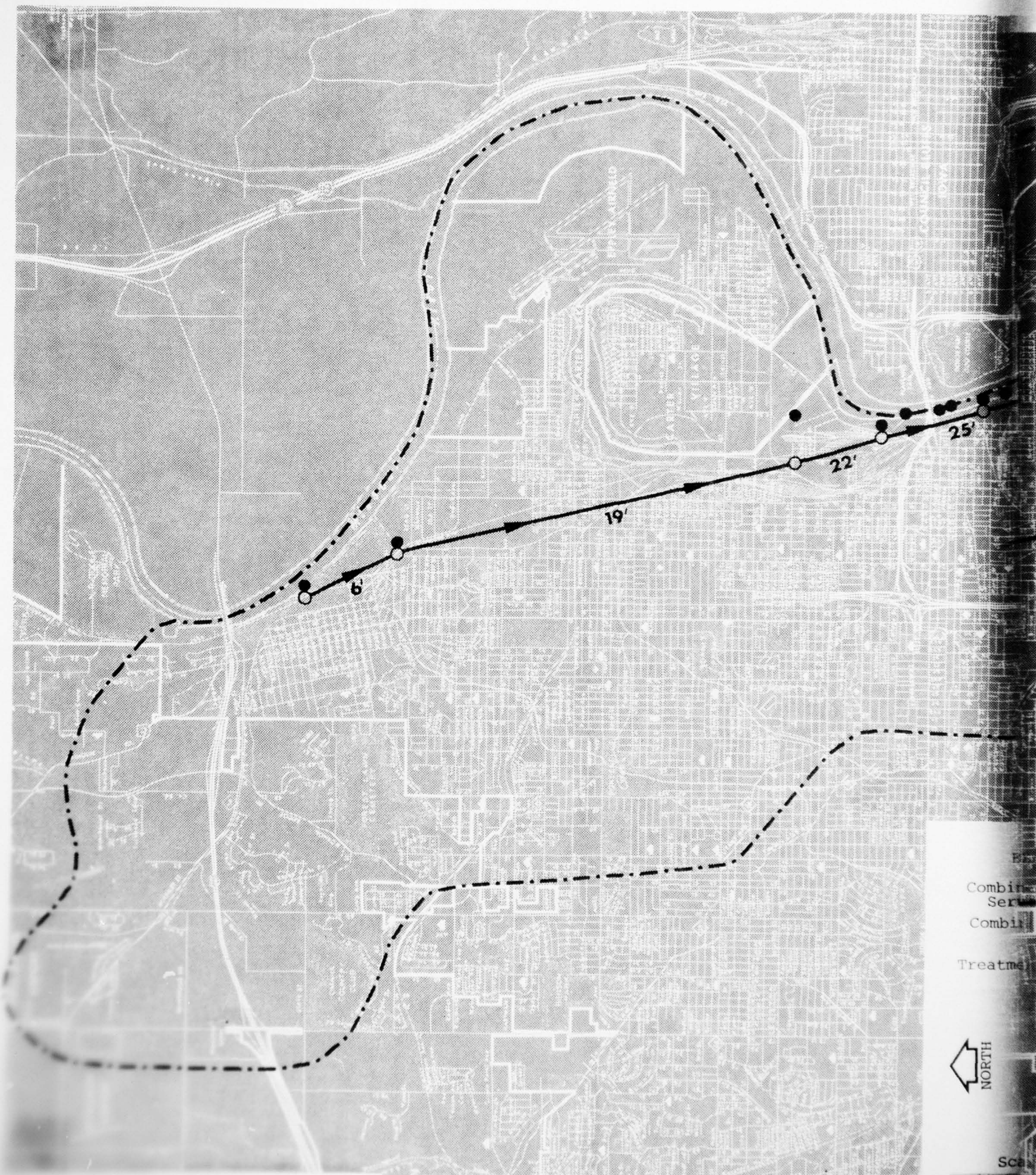
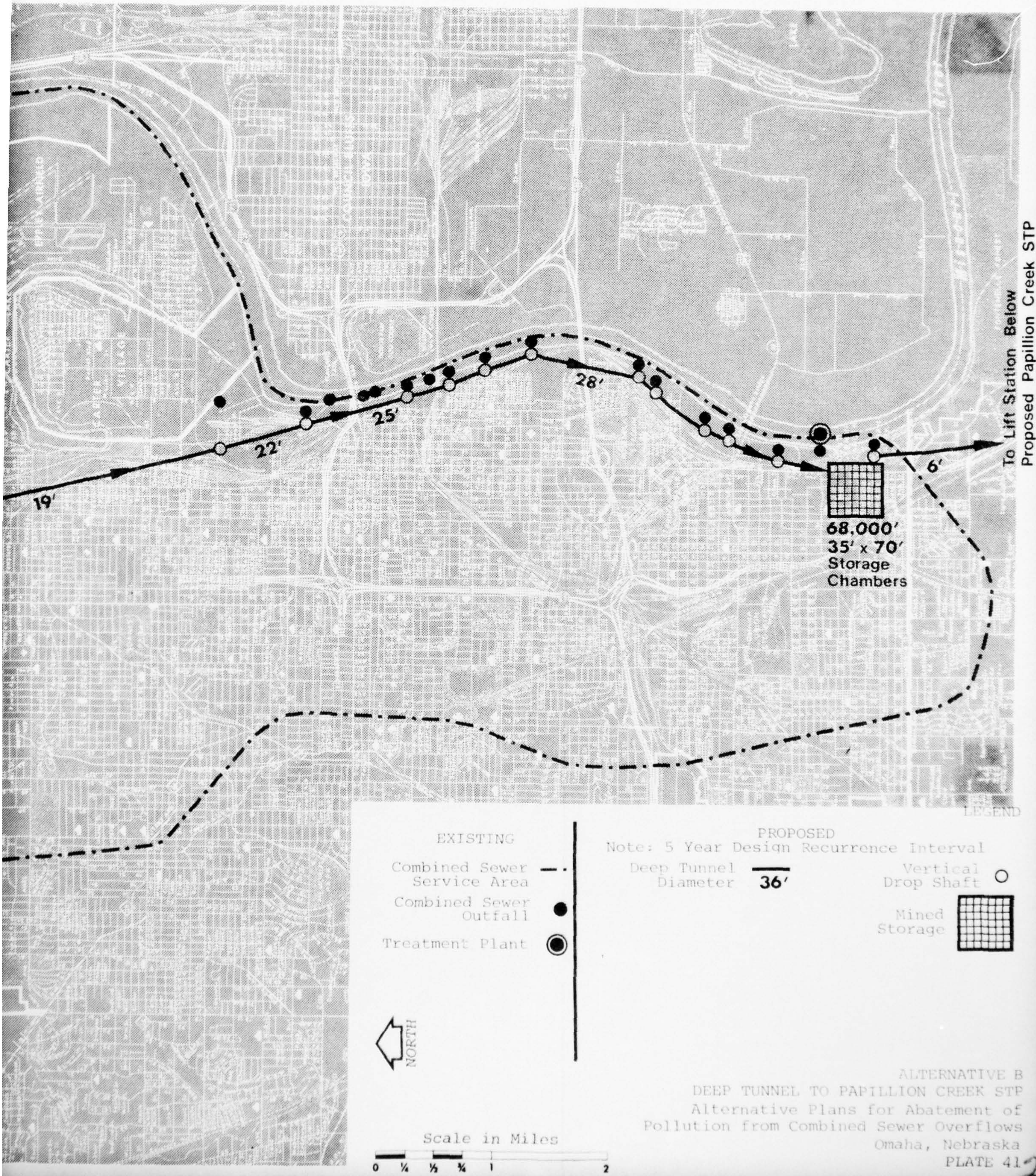


ALTERNATIVE NO. 5A
DEEP TUNNEL WITH MINED STORAGE
Alternative Plans for Abatement of
Pollution from Combined Sewer Overflows
Omaha, Nebraska
PLATE 40

2



Company - October 1974



ALTERNATIVE B
DEEP TUNNEL TO PAPILLION CREEK STP
Alternative Plans for Abatement of
Pollution from Combined Sewer Overflows
Omaha, Nebraska
PLATE 41

APPENDIX A

PLANNING AND ENGINEERING CRITERIA

Appendix A

PLANNING AND ENGINEERING CRITERIA

Planning and engineering criteria are needed to provide a consistent basis for evaluating and comparing alternative plans for abatement of pollution from combined sewers and storm run-off. The criteria which have been selected for this phase of the wastewater management study for the Omaha-Missouri drainage area are described in this Appendix.

The criteria used in formulating and comparing alternative plan concepts are as follows:

1. Population and land use
2. Hydrologic data
3. Water quality of combined sewage overflows
4. Economic evaluation criteria and procedures

Population and Land Use

Population projection by census tract were provided by the Metropolitan Utilities District and the Corps of Engineers. These projections were used as the basis for estimating the populations of each sewer service area for 1970, 1995, and year 2020. The population projections based on the MUD data are presented in Table A-1 for each sewer service area, as outlined in Plate 6. The alternative projections based on the Corpos of Engineers data for Growth Concept A are shown in parenthesis, also in Table A-1.

Table A-1

SERVICE AREA POPULATION IN

OMAHA-MISSOURI DRAINAGE AREA

Service Area	1970 Population	1995 Population		2020 Population	
Bridge Street	5320	7800	(6400)	9990	(9200)
Mormon Street	2100	2530	(2120)	2870	(2690)
Minne Lusa	47030	57190	(57450)	62255	(61805)
Grace Street	20635	22310	(26250)	25790	(27810)
Burt-Izard	23610	22315	(22420)	23245	(22070)
Leavenworth	21940	20385	(19210)	20620	(19150)
Gr. Missouri Ave.	11325	13135	(11385)	13680	(11740)
South Omaha	17600	15850	(15615)	15565	(14390)
Monroe Street	17945	19705	(15295)	21190	(15500)
Sub-Total	167,505	181,220	(176,145)	195,205	(184,355)
Carter Lake Area	5925	7515	(3275)	9720	(4000)
Total	173,430	188,735	(179,420)	204,925	(188,355)

Present and planned land use was used as a basis for determining the impact of the Riverfront Development Program (RDP) on the Omaha-Missouri River Sewerage System. The planned land use as proposed by the Riverfront Development Program is presented in Table III-1 and the locations of these areas are shown in Plate 5. The existing land use in these areas is based on the 1973 Omaha City Planning Department land use map and the 1971 revised U.S.G.S. map.

The change in land use was used to estimate the impact of the RDP on the wet weather and dry weather projected combined sewage flows. Wet weather combined sewage flows are affected by the amount of impervious area in a drainage area. Changes in land use that will increase the amount of impervious area will also increase the amount of runoff. The magnitude of

change in wet weather projected combined sewage flows was estimated by comparing the present ratio of impervious area to the projected ratio of impervious area. Dry weather flows may be affected as the water consumption varies with land use.

Hydrologic Criteria

Hydrologic analyses of the 18 sewer service areas in the Omaha-Missouri River sewerage system were made. Design discharge-frequency relationships, the 1,2,5,10, and 25 year design storm hydrographs, and the maximum 1,6, and 24 hour runoff volumes were determined for 17 of the service areas. For the Carter Lake area, only the maximum 1,6, and 24 hour runoff volumes were calculated.

Development of the design hydrologic discharges was based on existing land use patterns determined from aerial photographs of the study area flown in July 1972. Natural drainage area boundaries do not in all cases coincide with the sewer service boundaries; but officials of the Omaha Public Works Department stated that man-made boundaries imposed by the collection facilities contain the flows by ponding at the sewer inlets. It was assumed that the sewer service area boundaries are valid up to the 25-year recurrence interval rainfall event.

The peak discharge rate of the design storms for each sewer service area was computed by the Rational Formula $Q = C\ (I - F_{av})\ A$, where:

Q = peak runoff rate in cubic feet per second;

C = a runoff coefficient roughly representing the infiltration capacity of the pervious and impervious areas of the watershed. C is expressed as the composite

of the runoff co-efficients C_{per} and C_{imp} from the pervious and impervious areas of the watershed ($C_{comp} = C_{per} (P_p) + C_{imp} (P_i)$ wher P_p and P_i are the percent of pervious and impervious areas, respectively);

I = rainfall intensity in inches per hour for the critical time of concentration for each sewer service area;

F_{av} = available depression storage in inches per hour; and

A = drainage area expressed in acres.

The design storm point rainfall amounts were obtained from the rainfall-frequency-duration isopluvial maps contained in the U.S. Weather Bureau Technical Paper No. 40, "Rainfall Frequency Atlas of the United States", Map 1961. Shown on Table A-2 is a tabulation of the point rainfall amounts at Omaha, Nebraska for specific rainfall-duration-frequency storms. These point rainfall amounts were adjusted to obtain the average depth over the entire 33 square mile watershed contained in the 17 sewer service areas. The tabulation of reduced point rainfall amounts is shown on Table A-3. The runoff coefficient (C) for each area was weighted according to the watershed land use development, soil type and land slopes. The C values for the pervious areas were adjusted for specific frequency events by Bernards formula, $C = C_{100} \left(\frac{F}{100}\right)^x$ where:

C = the adjusted C for a specific frequency event F;

C_{100} = equals the chosen runoff coefficient for the 100-year event;

F = the frequency of the desired event in years; and

x = a coefficient equal to 0.18 for the vicinity of Omaha, Nebraska.

The tabulation of the basic runoff coefficients used is shown on Table A-4. The discharge frequency curves were developed for each sewer service areas by computing the peak discharge of the 1, 2, 5, 10 and 25 year storm events. The design storm hydrographs were developed using the following procecure: (1) the hydrograph peak discharges were computed using the rational formula; and (2) the hydrographs were sized and shaped using the time of concentration of each area and the distribution and volume of runoff for each respective area which were determined from the hypothetical hyetographs.

Overflow of combined sewage occurs whenever the rate of runoff exceeds the capacity of the diversion structure. The rainfall intensity that causes the runoff to exceed this capacity is estimated to be 0.10" per hour; and based on twenty years of precipitation records, occurs about 50 times per year.

The total amount of runoff that overflows to the Missouri River is estimated to about 5 billion gallons per year. This volume is based on a runoff coefficient of 0.60 for the drainage area of about 22,000 acres and a runoff of 14.3 inches (about one half of the yearly precipitation). About one half of the precipitation is either less then 0.10 inch per hour or does not runoff.

Table A-2

TABULATION OF DATA CORRESPONDING TO ENVELOPING CURVES OF ACCUMULATIVE RAINFALL-DURATION-FREQUENCY RELATIONS BASED ON POINT RAINFALL VALUES

Maximum Rainfall Duration In Hours	RAINFALL IN INCHES DEPTH CORRESPONDING TO VARIOUS AVERAGE FREQUENCIES AND DURATION IN HOURS[1/] Average Exceedence Frequency Interval In Years						
	1	2	5	10	25	50	100
(a) Maximum Accumulation of Rainfall							
0.25	0.76	0.91	1.19	1.40	1.62	1.82	2.04
0.50	1.06	1.27	1.66	1.95	2.25	2.53	2.84
1	1.35	1.62	2.11	2.47	2.86	3.21	3.60
2	1.55	1.86	2.45	2.84	3.30	3.70	4.15
3	1.68	2.04	2.64	3.07	3.54	3.97	4.46
4	1.77	2.16	2.78	3.24	3.72	4.20	4.76
5	1.85	2.26	2.94	3.40	3.90	4.40	4.98
6	1.92	2.35	3.07	3.55	4.06	4.56	5.15
24	2.52	3.05	3.95	4.60	5.02	5.98	6.67

1/ Point rainfall values for Omaha, Nebraska. Reference: U.S. Weather Bureau Technical Paper No. 40.

(b) Rainfall by 1-Hour Increments During Maximum 6-Hour Accumulation							
0-1	1.35	1.62	2.11	2.47	2.86	3.21	3.60
1-2	0.20	0.24	0.34	0.37	0.44	0.49	0.55
2-3	0.13	0.18	0.19	0.23	0.24	0.27	0.31
3-4	0.09	0.12	0.14	0.17	0.18	0.23	0.30
4-5	0.08	0.10	0.16	0.16	0.18	0.20	0.22
5-6	0.07	0.09	0.13	0.15	0.16	0.16	0.17
(c) Rainfall by 6-Hour Increments During Maximum 24-Hour Accumulation							
0-6	1.92	2.35	3.07	3.55	4.06	4.56	5.15
(d) Rainfall by 24-Hour Increments During Maximum 96-Hour Accumulation							
0-24	2.52	3.05	3.95	4.60	5.28	5.98	6.67

Table A-3

TABULATION OF DATA CORRESPONDING TO ENVELOPING CURVES

OF ACCUMULATIVE RAINFALL-DURATION-FREQUENCY RELATIONS

BASED ON AVERAGE RAINFALL VALUES

Maximum Rainfall Duration In Hours	RAINFALL IN INCHES DEPTH CORRESPONDING TO VARIOUS AVERAGE FREQUENCIES AND DURATIONS IN HOURS[1/] — Average Exceedence Frequency Interval, in Years						
	1	2	5	10	25	50	100
(a) Maximum Accumulation of Rainfall in Period Designated Column 2							
0.25	0.64	0.77	1.00	1.17	1.36	1.52	1.71
0.50	0.89	1.07	1.39	1.63	1.89	2.12	2.38

1/ Average rainfall amounts for a 33-square mile area at Omaha, Nebraska. Reference U.S. Weather Bureau Technical Paper No. 40.

1	1.13	1.36	1.77	2.07	2.40	2.69	3.02
2	1.37	1.65	2.15	2.32	2.93	3.29	3.69
3	1.49	1.81	2.34	2.73	3.15	3.55	3.96
4	1.59	1.94	2.52	2.91	3.35	3.78	4.22
5	1.68	2.05	2.67	3.09	3.54	3.99	4.48
6	1.74	2.13	2.79	3.23	3.69	4.14	4.68
24	2.41	2.92	3.79	4.41	5.06	5.74	6.40
(b) Rainfall by 1-Hour Increments During Maximum 6-Hour Accumulation							
0-1	1.13	1.36	1.77	2.07	2.40	2.69	3.02
1-2	0.24	0.29	0.38	0.45	0.53	0.60	0.67
2-3	0.12	0.16	0.19	0.21	0.22	0.26	0.27
3-4	0.10	0.13	0.18	0.20	0.23	0.26	
4-5	0.09	0.11	0.15	0.18	0.19	0.21	0.26
5-6	0.06	0.08	0.12	0.14	0.15	0.15	0.20
(c) Rainfall by 6-Hour Increments During Maximum 24-Hour Accumulation							
0-6	1.74	2.13	2.79	3.23	3.69	4.14	4.68
(d) Rainfall by 24-Hour Increments During Maximum 96-Hour Accumulation							
0-24	2.41	2.92	3.79	4.41	5.06	5.74	6.40

Table A-4

RUNOFF COEFFICIENTS

OPEN AND PARK AREA

1. Sandy soil, flat, 2% (100% Pervious) C_{100} = 0.10 - 0.15
2. Sandy soil, ave., 2-7% (100% Pervious) C_{100} = 0.15 - 0.20
3. Sandy soil, steep, 7% (100% Pervious) C_{100} = 0.20 - 0.30
4. Heavy Soil, flat, 2% (100% Pervious) C_{100} = 0.15 - 0.25
5. Heavy soil, ave., 2-7% (100% Pervious) C_{100} = 0.25 - 0.35
6. Heavy soil, steep, 7% (100% Pervious) C_{100} = 0.35 - 0.45

RESIDENTIAL DEVELOPMENT

7. Large Lot {30% Impervious, 70% Pervious; C_{imp} = 1.0 C_{per} = 0.10 - 0.45} C_{100} = 0.37 - 0.61
8. Normal (40% Impervious, 60% Pervious) (C_{imp} = 1.0 C_{per} = 0.10 - 0.45) C_{100} = 0.46 - 0.67
9. Dense (55% Impervious, 45% Pervious) (C_{imp} = 1.0 C_{per} - 0.10 - 0.45) C_{100} = 0.60 - 0.75

COMMERCIAL AND INDUSTRIAL DEVELOPMENT

10. Normal (80% Impervious, 20% Pervious) (C_{imp} = 1.0 C_{per} = 0.10 - 0.45) C_{100} = 0.82 - 0.89
11. Concentrated (95% Impervious, 5% Pervious) (C_{imp} = 1.0 C_{per} = 0.10 - 0.45 C_{100} = 0.95 - 0.97
12. Impervious Surface (100% Impervious) C_{100} = 1.00

Bernard's Equation: $C_F = C_{100} \left(\frac{F}{(100)}\right)^x$ for pervious areas.

x = 0.18 for Omaha, Nebraska

$C_{100} = 1.00 \times C_{100}$

$C_{50} = 0.88 \times C_{100}$

$C_{25} = 0.78 \times C_{100}$

$C_{10} = 0.66 \times C_{100}$

$C_5 = 0.58 \times C_{100}$

$C_2 = 0.49 \times C_{100}$

$C_1 = 0.44 \times C_{100}$

Water Quality of Combined Sewage

Wet-weather combined sewage flow may contain significant quantities of pollutants. The pollutants which are of the most concern in the Omaha system consist of BOD_5, suspended solids, ammonia, phosphorus, and fecal coliform because these are the significant pollutants upon which the desired levels of treatment are to be imposed. The concentrations of these pollutants in combined sewer overflow depend on a number of factors which include the duration and intensity of rainfall, the time period between rainfall events, the volume of runoff, the efficiency of street cleaning operations, and characteristics of the sewer system.

Data on the characteristics of Omaha's combined sewer overflows are not available. However, the water quality of the combined sewer discharges during several rainfall events in 1957 is reported in the 1958 report, "Interceptor Sewers Disposal" by Henningson, Durham and Richardson. In 1957, the main combined sewers discharged sewage directly to the Missouri River without treatment. These water quality data, presented in Appendix B, indicate that a wide range of BOD_5 and suspended solids concentrations are possible within the Omaha combined sewer system. Unfortunately, these data do not provide a reliable base for predicting the water quality of Omaha's combined sewer overflows because of the lack of additional information to explain the wide variation in sampling results.

A literature search was conducted to examine the issue of pollutant concentrations associated with wet-weather combined sewer flow. As would be expected for the phenomenon under considerations, very wide scatter of data is found. However, a definite trend exists. In almost all studies, the authors indicate high pollutant concentrations with the sewer system

"first flush", then a gradual decrease with time as the system is purged by the wet-weather flows, and an eventual establishment of a near-constant concentration of BOD_5 and suspended solids with storms of prolonged duration.

Neither the data from Omaha nor from the literature provide a reliable basis for predicting Omaha's combined sewer overflow water quality. For planning purposes the concentrations of the pollutants have been assumed to vary with time, as indicated in Plate 12, based on engineering judgement of the available data. The average concentrations of BOD_5, suspended solids, ammonia nitrogen, and phosphorus which are assumed for this study are presented in Table A-5.

Concentrations of BOD_5 and suspended solids in the overflows from Omaha's combined sewers are assumed to be 250 mg/l and 400 mg/l, respectively, during the "first flush" of runoff. These concentrations are assumed to decline to 50 mg/l of BOD_5 and 200 mg/l of suspended solids as the sewer system is purged. The concentration of ammonia nitrogen is assumed to range between 6 and 8 mg/l at the beginning of the runoff, but is assumed to drop by 50 percent as the rate of runoff tapers. The concentration of total phosphorus is assumed to range between 6 and 10 mg/l during the beginning of the runoff, but it also is assumed to drop by 50 percent as the rate of runoff tapers.

It is recognized that the "first Flush" concentrations may range quite widely in Omaha, depending on the rainfall intensity and duration,the time between rainfall events and the location and characteristics of the main sewer. The above values are assumed to represent an average of the possible concentrations and are therefore, not meant to be dependent upon specific factors such as rainfall intensity and duration.

Table A-5

ASSUMED AVERAGE CONCENTRATIONS OF POLLUTANTS IN COMBINED SEWER OVERFLOW, OMAHA, NEBRASKA

Pollutant	Average Value, mg/l
BOD_5	100
Suspended Solids	250
Ammonia Nitrogen	15
Phosphorus	4

Economic Evaluation Criteria and Procedures

Federal EPA requirements for planning studies state that plan recommendations should be based on a cost-effectiveness study. The criteria used in the economic evaluation and economic comparison of alternative plans for abatment of pollution from combined sewers are presented in this section.

There is an important distinction between an economic evaluation and a financial analysis. Financial analyses include the monetary receipts and disbursements associated with a project. An economic evaluation does not include a consideration of outstanding debt or debt retirement on a facility. In an economic evaluation the cost of a facility already constructed (sunk cost) is common to each alternative and is not directly considered. However, the economic evaluation does include the costs of replacing an existing component at the end of its remaining economic life. The effect of this procedure is to credit existing components through deferment of the replacement cost, whereas alternatives of new construction require immediate investments.

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WATER AND RELATED LAND RESOURCES MANAGEMENT STUDY. VOLUME V. SU--ETC(U)

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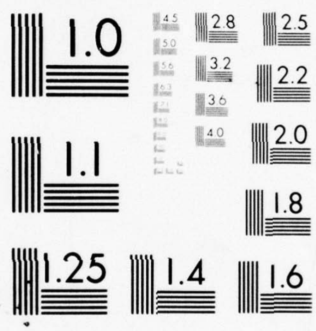

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

Following the principles of engineering economic analyses, no escalation of costs for construction, operation, maintenance, or replacement was considered. Costs for system components are based on March 1974 costs. Estimated land costs are presented in Table A-6.

Economic Criteria

The criteria for economic evaluation consist of the planning period, interest rate (the time value of money), the economic life of facilities, the design period for new facilities, and the selection of conversion factors used in an economic evaluation. The economic criteria are used to evaluate the alternatives for wastewater management and to compare these alternatives on the basis of total present worth cost.

Planning Period. A planning period from the present to 1995 was selected in the analysis of alternatives for cost-effectiveness. The present worth cost of an alternative includes the capital costs of all construction required within the planning period and the replacement of those facilities between the present and year 2020.

Interest Rate. The interest rate used in economic evaluation should reflect the best estimate of the costs incurred by investors in foregoing investment in other sectors of the economy. Normally rates of 4 to 10 percent are used for regional planning. An evaluation was made to determine the efcect of comparing the cost of different alternatives for interest rates of 4, 7, and 10 percent. The effect was negligible. Based on this evaluation and current recommendations of the Federal EPA, and interest rate of 7 percent was selected for use.

Economic Life. For purposes of economic evaluation, economic lives of 50 years for sewers and force mains and 25 years for pumping stations, lift stations, and sewage treatment facilities were selected. The 25-year economic life for a sewage

Table A-6

ESTIMATED LAND COSTS

General Location	Estimated Cost/Acre 1/	Owners/Acre 2/
1. North of Carter Lake & west of Eppley Airfield	$25,000	0.20
2. Triangle below Carter Lake	25,000	0.50
3. Around sewage treatment plant	45,000	2.0
4. North of Council Bluffs, across Missouri River from Eppley Airfield	12,000	2 3/
5. East side of levee and south of Council Bluffs	10,000	2 3/
6. West of levee, from Carter Lake to sewage treatment plant	40,000	2.0
7. General - 55th Street west to to 30th Street east- north of Dodge Street	75,000	3.0
8. Western area - south of Dodge Street	40,000	2.0
9. Land east of levee in Nebraska from:		
a. Eppley Airfield to Locust St.	10,000	
b. Locust to Hamilton	25,000	
c. Hamilton to Bancroft	12,000	
d. Brancroft to "F"	3,000	
e. "F" to treatment plant	12,000	

1/ Omaha District, Corps of Engineers
2/ Acquisition costs are approximately $2000/owner
3/ Owners in area

treatment plant is based on the cost of plant components relative to the total cost and anticipated useful life for the various components. If the end of a facility's economic life occurs within the economic evaluation period, the cost of replacement is included in the present worth analysis.

Design Period. New sewers, force mains, and storage basins are sized for year 2025. Treatment facilities are sized on the basis of projected loadings for the next 20 years.

Factors Used in Economic Evaluation. Two factors are commonly used in the economic comparison of alternatives. These are the single payment present worth factor (PWF) and the series present worth factor (SPWF). The single payment PWF converts a future investment cost to equivalent present value and is used in determining the present value of future investment to replace existing facilities. The SPWF converts a series of uniform annual payments such as operation and maintenance costs to an equivalent annual lump sum cost at the beginning of the period.

APPENDIX B

WATER QUALITY OF COMBINED SEWER OVERFLOW AND SEPARATE STORM SEWER DISCHARGE

Appendix B

WATER QUALITY OF COMBINED SEWER OVERFLOWS AND SEPARATE STORM SEWER DISCHARGE

The water quality of combined sewer overflows and storm water discharge may vary from municipality to municipality, discharge point to discharge point, and rainfall. Because of this variance, a search of the literature on the subject of combined sewer overflows and storm water discharges was conducted to gather some insight on the water quality of combined sewer overflows and separate storm sewer discharges in the Omaha-Missouri River sewerage system.

Water Quality of Combined Sewer Overflows

The only reported sampling program on the Omaha-Missouri River combined sewer system is presented in the 1958 report, "Interceptor Sewers and Sewage Disposal" by Hennington, Durham, and Richardson. The water quality of the combined sewer discharge was determined during several rainfall periods prior to the construction of the present primary plant. The results of the tests, presented in Table B-1 for non-proportional 24-hour composite samples, indicate a wide variation of BOD_5 concentrations depending upon location and rainfall event.

Burn, et. al.,[1/] studied combined sewage flows from an area serving about one third of Detroit: about 22,000 acres and 400,000 people. The area varied from residential to commercial to light industrial in nature. They found that the BOD of the combined runoff declined appreciably with time, the concentration being about 200 mg/l at the beginning of a storm and declining to 90 or 100 mg/l for flows 6 hours or more after the beginning of the storm.

Table B-1

TABULATION OF ANALYSES ON SEWAGE SAMPLES COLLECTED

OMAHA, NEBRASKA[1/]

Location	Average Flow (MGD)	1957 Date	pH	BOD (mg^5/l)	Suspended Solids (mg/l)	Volatile Suspended Solids (mg/l)
Douglas St. Outlet	Rain[2/]	11/16	8.6	860	2468	1894
Jones St. Outlet	Rain[2/]	9/10-9/11	7.0	30	35	30
Leavenworth St. Outlet	Rain[2/]	9/18-9/19	6.6	340	275	250
Leavenworth St. Outlet	Rain[2/]	9/20-9/21	7.0	325	290	205
Hickory St. Outlet	Rain[2/]	9/10-9/11	7.4	80	110	80
Missouri Ave.	Rain[2/]	10/22-10/23	7.7	420	70	47

1/ "Interceptor Sewers and Sewage Disposal", Volume 2, Henningson, Durham, and Richardson, 1958.
2/ Flow not measured during rainfall periods

Suspended solids were found to decline only slowly following the beginning of storm runoff. The concentration averaged from 275 to 300 mg/l the first 6 to 12 hours of storm runoff, and then declined to about 200 mg/l. Ammonia nitrogen is found in higher concentrations in the early hours of a storm than the later. It averaged about 8 mg/l at the start of storm runoff and declined to 5.2 mg/l after storm runoff had continued over 12 hours.

For each of these three parameters of pollution, it was found that the concentrations were higher during the spring months (April and May) than during the fall months. The summer months were generally higher than the fall. For BOD_5

most of the change in concentration that occurred between spring and summer was greater than between summer and fall. For ammonia, the change occurring between summer and fall was greater; and for suspended solids, the change in concentration was more or less uniform with a portion occurring with each change of the seasons.

Benzie and Courchaine[2/] had reported earlier on some aspects of combined sewer overflow quality in Detroit. Their measurements of suspended solids with a mean of 150 mg/l were slightly lower than those found by Burm, et. al. Their average concentration of NH_3-N, 3.25 mg/l, was also lower.

Their survey lasted from early May, 1963 through late October, 1964. However, samples were only composited for ammonia and suspended solids determinations between late March and late September, 1964, which corresponds well with the season of the year included in the study of Burm, et. al.

Burgess & Niple, Ltd.[3/] reported on a study of combined sewers in Bucyrus, Ohio. Their sampling program was of limited duration, and covered only 5 storm events. They found an average BOD of about 130 mg/l, suspended solids of about 480 mg/l and ammonia nitrogen of 2.3 mg/l.

They found that the BOD concentration declined during the course of a storm event, being perhaps 250 mg/l at the start of a storm, and declining to concentrations of 200 mg/l at 50 minutes, 100 mg/l at two and a half hours, and 50 mg/l or less when overflow continued beyond four hours. They also found that the BOD could be improved greatly by plain sedimentation, at least during the early portion of a storm. When the BOD of the combined sewage was about 200 mg/l, one half hour of sedimentation would reduce the BOD to below 100 mg/l, one hour

to less than 50 mg/l, and periods over two hours would reduce the BOD to well below 50 mg/l.

Burgess & Niple reported that the amount of BOD that runs off of a given unit of land area is a function both of the amount of precipitation and of the time elapsed from the last rainfall. Pravoshinsky and Gatillo[4/] reported on studies of storm runoff in Minsk, U.S.S.R. extending from 1950 through 1966 (17 years). They, too, found a relationship between the time interval between storms, the intensity of the storm, and the pollution load.

Inreasing pollution load with increasing time from the preceeding runoff combined with concentrations that decline with flow duration were also found by Davidson and Gameson[5/]. These investigators had studied successively three separate areas (Northampton, Bradford, and Brighouse) in England. The areas studied were primarily residential, were studied for at least two years each, and had areas of 167, 229, and 597 acres. The authors suggest average concentrations of suspended solids of 400 mg/l, 60 mg/l BOD_5, and 4 mg/l ammonia nitrogen.

Davidson and Gameson indicate that concentrations of BOD_5, suspended solids, and ammonia all decline from maxima at the start of storms to minima within an hour to an hour and a half. The maximum for Northampton was about 350 mg/l, and the minima averaged about 60 mg/l for 5-day BOD. Suspended solids began with concentrations near 600 to 800 mg/l and declined to about 200 mg/l. Ammonia concentrations declined from about 20 at the start of a storm to about 4 during the last of the runoff period.

Diaper and Glover[6/] reported that the BOD in combined sewage from an 11 acre area in Philadelphia average 112 mg/l for 26 events. During these same events, an average suspended

solids of 184 mg/l was found.

Guarino, et. al.[7/] reported on a sampling program covering a portion of Philadelphia served with combined sewers. The population was about 173,000 people in an area of 5,400 acres. Difficulties were experienced with the automatic proportional sampling equipment, so that only grab samples were available. Based on those samples collected at known times and measured flow, it was determined that the "volume of runoff"-"total load of BOD" relationship is approximately as tabulated below.

Table B-2

RELATIONSHIP BETWEEN RUNOFF VOLUME AND BOD LOAD

Percent of Total Volume of Runoff	25	50	75	100
Percent of Total BOD Load	50	73	90	100

Eckoff[8/] presented data collected in San Francisco where sewers tend to be laid on relatively steep slopes which minimize the deposition of solids. The average BOD of the combined sewage in San Francisco was found to be 36 mg/l; suspended solids of 224 mg/l were found; and total nitrogen was found to be 4 mg/l.

Friedland, et. al.[9/] reported on additional studies on six basins in San Francisco totaling 4,380 acres and having 130,000 inhabitants. Each of the individual basins is predominantly residential in nature. The collection of data extended from November, 1966, through February, 1970, although only one basin was sampled on any one occasion.

They conclude that combined sewage typically goes through the following phases after runoff begins:

First Phase: characteristics nearly the same as domestic sewage.

Second Phase: of short duration pollutant concentrations may exceed domestic sewage, and

Third Phase: strength declines to that of dilute domestic sewage. In San Francisco, concentrations during the second phase may be twice that of domestic sewage, while during the third phase, they may be 10% to 25% of the domestic values.

Lager et.al.[10/] reported on the development of a computer model which was calibrated with the aid of data from San Francisco. The measured data for BOD concentrations indicate the BOD is below 50 mg/l for extended periods during the tail end of the runoff period.

Marske[11/] tested screening methods for treatment of combined sewer overflows in Portland, Oregon. Water quality of combined sewer overflows from an area of 25,000 acres was determined and reported as shown in Table B-3.

Table B-3

COMBINED SEWER OVERFLOW IN PORTLAND, OREGON

Characteristic	Mean Value (mg/l)	Standard Deviation
BOD_5	105	$\pm$ 25
Suspended Solids	146	$\pm$ 59
Ammonia Nitrogen	5.1	$\pm$ 1.4

The combined sewage characteristics at Sacramento, California reported by Envirogenics Company[12/] are shown in Tabel B-4.

Table B-4

COMBINED SEWAGE CHARACTERISTICS AT SACRAMENTO, CALIFORNIA

Item	Concentration, mg/l*		
	At Start	3 Hours Later	12 to 18 Hours Later
BOD	283	148	155
Suspended Solids	230	106	146

* Each value represents averages for 5 or 6 events between December, 1968 and May, 1969.

Mason[13/] reported on a program to treat combined sewage in Milwaukee, Wisconsin from a 495 acre residential area. The quality of the sewage before treatment was reported as presented in Table B-5.

Table B-5

COMBINED SEWAGE CHARACTERISTICS AT MILWAUKEE, WISCONSIN

Flow During	Concentration, mg/l		
	BOD	Suspended solids	Kjeldahl N
First 20 to 60 Minutes	186 ± 40	522 ± 150	17.6 ± 3.1
Time Over 60 Minutes	44 ± 10	166 ± 26	5.5 ± 0.8

Vilaret and Pyne[14/] reported that storm sewage in Atlanta, Georgia, has an average BOD of 143 mg/l.

Filippi and Shih[3/] reported on a program testing the quality of combined sewer overflows in Washington, D.C. They examined two drainage basins, one of 110 acres and the other of 265 acres. They found the following concentrations to be representative of the quality: BOD_5, 71 mg/l, suspended solids, 622 mg/l, and NH_3-N, 1.5 mg/l.

Davis and Borchardt[22/] reported on a program for a combined sewer overflow abatement plan for Des Moines, Iowa. Laboratory analysis of the discharges sampled in Des Moines showed combined sewer overflows to have concentrations of BOD_2, 72 mg/l; suspended solids, 329 mg/l; and, NH_3-N, 4.7 mg/l.

The reports discussed above are summarized in Table B-6.

Table B-6

SUMMARY OF COMBINED SEWAGE QUALITY

Community	Average Concentration of Pollutant, mg/l			
	BOD	Suspended Solids	NH_3-N	Area Acres
Atlanta, Georgia[14/]	143			
Bucyrus, Ohio[3/]	130	480	2.3	1,009
Chicago, Ill.[21/]	50	275	6*	
Des Moines, Iowa[22/]	72	329	4.7	4,000
Detroit, Mich.[1/]	150E	250E	6E	22,000
Detroit, Mich.[2/]		150	3.2	
Milwaukee, Wisc.[13/]	75E	200E		495
Northampton, England[5/]	80	400	4	
Philadelphia, Penn.[6/]	112	184		11
Portland, Oregon[11/]	105	146	5.1	25,000
Sacramento, Calif.[12/]	150E	140E		
San Francisco, Calif.[8/]	36	224	4	
San Francisco, Calif.[9/]	30%**	30%**	30%**	4,380
San Francisco, Calif.[10/]	50			
Washington, D.C.[15/]	71	622	1.5	

* For runoff greater than 1,000 acre feet.
** Values given are percentage of dry weather values.
E Denotes values which have been estimated.

As can be noted from a review of the preceding table, the reported average values of BOD_5, suspended solids, and ammonia nitrogen concentrations in combined sewer overflow varies from system to system. However, for planning purposes, the concentrations listed in Table B-7 are assumed to represent the average values of BOD_5, suspended solids, and ammonia nitrogen in combined sewer overflows in Omaha, Nebraska.

Table B-7

ASSUMED CHARACTERISTICS OF COMBINED SEWER OVERFLOWS IN OMAHA, NEBRASKA

Characteristic	Average Concentration (mg/l)
BOD_5	100
Suspended Solids	250
Ammonia Nitrogen	5

Storm Water Discharge

Storm waters unmixed with sanitary sewage were reported with a range of BOD from 17 to 236 mg/l and suspended solids from 600 to 14,500 mg/l by the American Public Works Association in "Water Pollution Aspects of Urban Runoff."[16/] The report presents a summary of sampling programs conducted in other cities around the world for the most part.

Vilaret and Pyne[14/] reported that based on their studies in Atlanta, Georgia, the BOD of separate storm runoff (and therefore, the storm flow component of combined runoff) varies in proportion to the rate of flow. They found that over half of the annual BOD load from storm runoff in Atlanta is due to storms with a frequency of two weeks or less.

Filippi and Shih[15/] reported on a sampling program for a 265 acre area in Washington, D.C., served by separate storm

sewers. They found BOD concentrations to range from 3 to 90 with an average of 19. Suspended solids were found to range from 130 to 11,300 with an average of 1,700. Nitrogen was not reported.

Soderholm[17/], et. al., drawing on experience in Stockholm, Sweden, conclude that runoff from areas with separate storm sewers can contribute significant amounts of oils to receiving streams when the areas served convey a lot of traffic. They too found generally low values of BOD, with a maximum of about 85 mg/l, but an average of about 20 mg/l.

AVCO[18/] reported on testing of separate storm sewers in Tulsa, Oklahoma and finding the average BOD to range from 8 to 18 with an average for all 15 areas sampled of 12 mg/l. Suspended solids were found to range from 84 to 2,050 with an average of 370 mg/l. The seven residential areas BOD averages ranged from 8 to 18 with an average of about 12 mg/l. The three commercial areas had average BOD's ranging from 8 to 14 and an average of about 11 mg/l. There were three areas classified as being primarily industrial, and the average BOD's of the runoff from these areas was 13 mg/l. The remaining two areas were in use as an airport and as a golf course, and the average BOD from these two areas was about 9 mg/l (the golf course being somewhat the higher of the two).

The seven residential areas sampled by AVCO ranged from 64 acres to 550 acres in area with an average area of about 250 acres. The testing program extended for a year during which 30 storms occurred. Samples were taken to characterize the 16 parameters being investigated.

Weibel[19] discussing runoff from 27 acres of residential land in Cincinnati, indicates that the average concentration of BOD was 17 mg/l and of suspended solids 227 mg/l. He had found[20] evidence that the concentration of suspended solids varied with the rate of storm water runoff, peaking when the rate of runoff peaks. For this small watershed, peak runoff rates occur when the rainfall intensity peaks, and any given storm may have more than one period of peak intensity, or a period of peak intensity which is not early in the storm.

Burm, et. al.,[1] report that the separate sewer system in Ann Arbor, Michigan, which they investigated had a drainage area of 3,800 acres. For this area, a storm runoff which lasted more than an hour was unusual in occurrence. Their investigation lasted for most of calendar 1965, but the data for the first four months of the year were not included because of problems with the collection system. The annual mean BOD concentration which they found was 28 mg/l, with insignificant variation during the months reported. The suspended solids had an annual average concentration of 2,080 mg/l from this essentially residential area, with possibly a heavier concentration in the spring. Ammonia nitrogen was uniform at 1.0 mg/l through the sampling period.

Benzie and Courchaine[2] previously reported on an earlier phase of the same investigation involving the 3,800 acres of Ann Arbor with separate sewers. They had found ammonia nitrogen of 0.5 mg/l and suspended solids of 1,280 mg/l.

Evirogenics[12] reported on separate storm quality in Sacramento, California. They indicate that the average quality of the runoff varied with time approximately as shown in Table B-8.

Table B-8

TIME VARIATION OF RUNOFF

Parameter	Hours After Beginning of Storm		
	0	3	12 to 18
BOD	127	132	82
Suspended Solids	58	95	48

Davis and Borchardt[22/] reported on separate storm water quality in Des Moines, Iowa. Laboratory analysis showed the concentration of storm water discharges to be BOD_5, 53 mg/l; suspended solids, 448 mg/l; and, NH_3-N, 1.78 mg/l.

These several studies are summarized in Table B-9 which follows.

Table B-9

SEPARATE STORM WATER QUALITY

Community	Concentration, mg/l							Area, Acres
	BOD_5			Suspended Solids			NH_3-N	
	Min.	Avg.	Max.	Min.	Avg.	Max.	Avg.	
Atlanta, Ga.[14/]	-	17	-		-		-	-
Ann Arbor, Mich.[1/]	-	28	-	1130	2080	2820	1.0	3800
Ann Arbor, Mich.[2/]	-	-	-	900	1280	2062	0.5	3800
Chicago, Ill.[16/]	-	77	-		-	-		-
Cincinnati, Ohio[19/20/]	1	17	173	5	227	1200	-	-
Des Moines, Iowa[22/]	-	53	-	-	448	-	1.8	4000
Sacramento, Calif.[12/]	25	110	283	3	67	211	-	-
Stockholm, Sweden[17/]	3	20	85	-	-	-	-	-
Tulsa, Okla.[18/]								5586
Residential	1	12	39	136	-	332	-	1815
Commercial	2	11	27	169	-	401	-	1293
Industrial	3	13	29	195	370	2052	-	1992
Open	6	9	23	89		445	-	486
Washington, D.C.[15/]	3	19	90	130	1700	11300	-	265

It should be noted that in most instances the values reported represent averages of several measurements. In some cases, the observed variation in quality is rather extreme. In Sacramento, for example, all the measurements of storm water quality reported here were made from one drainage area, and BOD varied from 25 mg/l to 283 mg/l. The data reported for Tulsa, on the other hand, fell in a much narrower range, even though a total of 15 drainage basins were sampled for one year.

Given the variablility of sewer design velocities from one system to another; the variation in rainfall intensity, duration, and distribution during the year; the variability in sizes of systems it is difficult to draw any conclusion from the available data. However, for planning purposes, the concentrations in Table B-10 are assumed to represent the average values of BOD_5, suspended solids, and ammonia nitrogen in storm sewer discharges in Omaha, Nebraska.

Table B-10

ASSUMED CHARACTERISTICS OF

STORM SEWER DISCHARGE IN OMAHA, NEBRASKA

Characteristic	Concentration (mg/l)
BOD_5	18
Suspended Solids	400
NH_3-N	0.5

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APPENDIX C

GEOLOGY

Appendix C

GEOLOGY

SUMMARY

About 2000 feet of a sedimentary rock sequence overlies Precambrian granite in the Omaha area. The rock sequence comprises interbedded shales, sandstones, limestones and dolomites ranging in age from Pennsylvanian to Cambrian. Stratigraphic intervals apparently adequate for tunneling and other underground structures include the Pennsylvanian Kansas City Group of interbedded shale and limestone at depths estimated to range up to 105 feet, and a massive dolomite of Mississippian age at depths estimated to range between 450 to 700 feet. The massive dolomite would be a more suitable rock for large underground excavations.

Alluvium in the Missouri River flood plain consists of varying admixtures of clay, silt, sand and gravel. Thickness of the alluvium is in excess of 100 feet. The alluvium is very permeable and largely saturated. The construction of underground structures in the alluvium would be very difficult.

Overburden on the bluffs adjacent to the river flood plain consists mostly of glacial till and loess, in thicknesses ranging up to and exceeding 100 feet.

REGIONAL GEOLOGY

The Omaha area comprises a broad loess-mantled upland till surface bisected by the valley of the Missouri River. In the bedrock subsurface the area lies on the northeast flank of the Nehawka Arch, a dominant structural feature. Rock exposures

in the area are part of the Pennsylvanian Kansas City Group. About 2000 feet of unexposed sedimentary rocks overlie the Precambrian igneous and metamorphic rocks beneath the Omaha area. These sedimentary rocks range in age from Cambrian through Pennsylvanian.

The only rock exposures in the Omaha area are confined to the Missouri Series of Late Pennsylvanian age. The Cretaceous Dakota Sandstone may locally occur in the subsurface overlying the Pennsylvanian rocks. Unconsolidated deposits of loess, glacial till, alluvium and slope wash, all of Quaternary age, mantle the bedrock formations.

PROJECT GEOLOGY

Stratigraphy

Surficial Deposits. The surficial deposits in the project area consist primarily of channel and flood plain alluvium and alluvial terrances. This alluvium comprises in large part admixtures of gravel, sand and silt with only local concentrations of clay. The clay deposits are most prominent in relic channels of the Missouri River. The clay in these areas may range up to 40 ft. in thickness. The thickness of the alluvium ranges in excess of 100 ft. The terrance deposits occur along the edges of the Missouri River flood plain, abutting the bluffs, and in the tributary stream valleys. These deposits range to about 30 ft. in thickness. The bluffs above the Missouri River are mantled mostly by loess and glacial till. In some local areas bedrock is exposed along the bluffs.

Minor alluvial-fan and slope-wash deposits occur throughout the area. These materials are primarily sand and silt, derived in large part from the loess.

Bedrock. The most prevalent bedrock unit underlying the Omaha area is the Kansas City group of Pennsylvanian age. There are also local occurrences of the younger Lansing Group. The average subsurface thickness of the Kansas City Group is about 180 ft. Due to erosion it may be thinner in local areas. This Group consists of relatively pure limestone, with interbeds of cherty and argillaceous limestone, claystone, and siltstone. Much of this rock sequence is thin bedded. The base of the Kansas City rocks is marked by a thin, 4 to 10 ft. thickness of red shale and sandstone. Some of the limestone beds in the Kansas City Group contain as much as 98% soluables. Therefore, the presence of solution cavities is a possibility. None are described in the literature and none were reported in the drill logs readily available, but the possibility of their existence cannot be ignored.

Underlying the Kansas City Group is the Marmaton Group, consisting of about 120 ft. of interbedded limestone and shale.

The Cherokee Group, consisting of about 300 ft. of thin bedded shales, siltstones and sandstones, with minor beds of coal and limestone occurs below the Marmaton Group. The Cherokee Group directly overlies the Mississippian rocks.

Massive limestone and cherty dolomite of Mississippian age underlies the Cherokee Group. These rocks reportedly have a thickness of about 200 ft. and occur about 450 to 700 ft. below ground surface.

The above rock units comprise an aggregate estimated thickness of about 500 ft. About 1500 ft. of sedimentary rocks, ranging in age from Devonian to Cambrian, occur between the Mississippian rocks and granite (see Plate C-1). The top of the Precambrian rocks in the Omaha area generally occur between

900 and 1300 ft. below sea level.

Structure

The bedrock units dip generally to the east and southeast at about 2 to 3°. This dip is interrupted by a probable extension of the Humboldt Fault. Some authors refer to this feature in the Omaha area as the Humboldt Monocline. In any event, this is a zone of faulting or a flexure, with the east side structurally lower. Reportedly there are about 35 feet of flexure or displacement at the bedrock surface, increasing with depth.

ENGINEERING GEOLOGY

Surficial Deposits. Foundations for engineering structures in the surficial deposits would largely comprise loess, or loess-derived materials, and alluvium. The loess is an inferior foundation material. It is subject to differential settlement when wetted or saturated and commonly has low shear resistance. The natural densities of the loess in the Omaha area range between 80 and 95 pcf. Some large buildings in the Omaha area required piles, penetrating the loess to bedrock, for support. The sandy alluvium contains areas of silt and clay, especially the clay deposits that occur in relic channels of the Missouri River. The physical properties of these materials are not presently defined in the area. Few structures in the area, however, show distress due to an inferior foundation.

Slope stability in the surficial deposits is highly variable. Major slump blocks occur along the Missouri River bluff. These blocks include loess as well as the underlying till. Undercutting the bluff slope or saturation of the till may have provided the mechanism for the slumping. The loess stands

vertically in most dry excavations. Moisture, however, tends to reduce slope stability causing sloughing and/or slumping. The glacial till is compact and brittle when dry and can hold relatively steep slopes. However, the silty and clayey matrix material becomes soft and sticky when wet and causes difficulties with excavation equipment. Cut slopes in the alluvium are highly variable dependent on the materials. Sands and gravel lenses would stand on moderately steep slopes.

Surface reservoirs sited on alluvial materials in the flood plain would require detailed site investigations. The deposits are highly permeable and the water table is high.

Bedrock. Tunneling is technically feasible in both the Kansas City Group rocks and in the Mississippian rocks.

The rocks of the Kansas City Group comprise mostly thin-bedded strata. The Winterset Limestone, however, in the lower part of the Group, reportedly comprises about 30 ft. of fairly massive limestone, and would be favorable for tunneling. The Marmaton Group is very shaley and extensive vertical fracturing may be present in the upper part. Any tunneling in this Group would probably require moderate support and would also require lining. The Cherokee Group is very thin bedded and shaley. If the Cherokee Group is typical of coal-bearing formations, then tunneling would be extremely difficult. Typical coal-bearing sequences commonly contain soft, weak rock types, such as clay shales. This unit is not favorable for tunneling. A shallow tunnel plan would probably penetrate rocks of both the Kansas City and Marmaton Groups.

The Mississippian rocks consist of massive limestone and dolomite, which is favorable for tunneling. Depths to these units would range between 450 and 700 ft. These rocks would

probably require little support and no lining.

The Humboldt Monocline must be considered a geologic hazard for any tunnel. Severe support problems and high groundwater inflows may occur in this zone. Extensive grouting and possibly lining will be required to control groundwater inflow.

Mined storage is being considered as one alternative for types of storage. The Mississippian dolomite offers the best potential horizon at a reasonable depth. Mined storage in and near the Humboldt Monocline should be avoided.

Groundwater

Little information is available concerning exact groundwater levels in the project area. However, it can be generally assumed that the levels will be at a depth of a few feet in the Missouri River alluvium and at a greater depth in the overburden and loess capping the bluffs. It can be assumed that bedrock in the project will be below the water table. High inflows would occur in rock excavations adjacent to, and immediately below, the Missouri River. Extensive grouting would probably be required in the Kansas City and Marmaton Group rocks to control groundwater inflows.

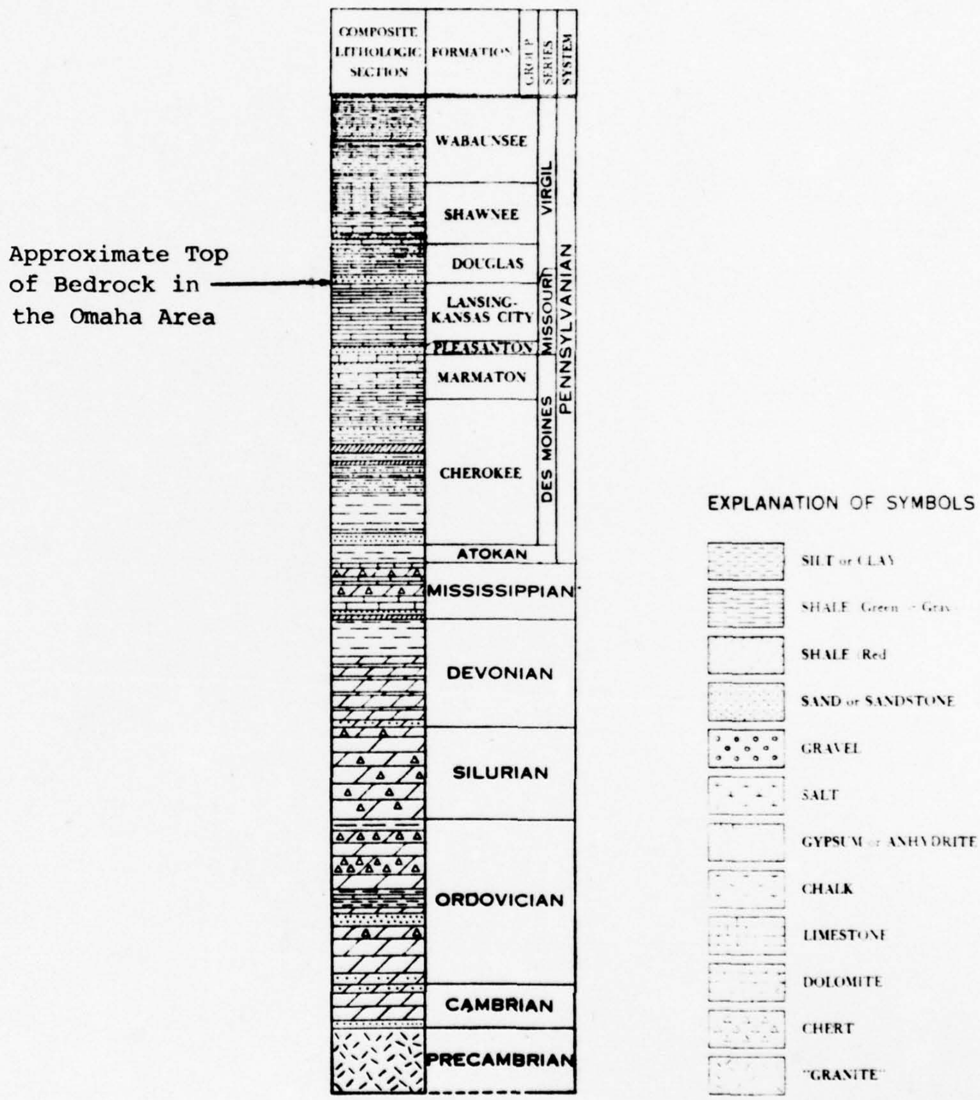
Flooding and Erosion

All surface structures located in the flood plain will be subject to flooding. Also some proposed surface structures are located in areas of active erosion by the Missouri River. In general the Missouri River is migrating to the west. Surface structures placed in such areas may need protective structures or riprap to prevent erosion.

Recommendations for Subsurface Exploration

Core holes should be drilled to determine the exact location, hydrologic and physical features of the Humboldt Monocline, and the adjacent rocks. Core holes are needed to further identify stratigraphic intervals that may be favorable to rock tunneling and mined storage in the Kansas City Group and to identify the subsurface configuration of the Mississippian rocks.

Soil sampling and physical testing will be necessary to evaluate the foundation materials along conveyance tunnels located in the overburden. Soil sampling and testing should also be done at sites of proposed treatment plants to evaluate foundation conditions, and in areas of surface storage, to check for excessively permeable foundation materials.



Source :
From Bedrock Geologic Map of Nebraska,
Nebraska Geological Survey.

GENERALIZED COLUMNAR SECTION - OMAHA AREA
Alternative Plans for Abatement of
Pollution from Combined Sewer Overflows
Omaha, Nebraska

APPENDIX D

SCREENING AND DISSOLVED-AIR FLOTATION SYSTEM

Appendix D

SCREENING AND DISSOLVED-AIR FLOTATION SYSTEM

The sixth alternative that was considered for abatement of the pollution from the combined sewer overflows is the installation of permanent screening/dissolved-air flotation treatment facilities at each of the 20 combined sewer outlets that are located along the west bank of the Missouri River.

The screening/dissolved-air flotation units, which are manufactured by Envirex, Inc. (a subsidiary of Rexnord, Inc. of Milwaukee, Wisconsin), consist of screen chambers and flotation chambers that are operated in series. The combined sewer overflow that will be treated by each of the screening/dissolved-air flotation units is diverted into the screen chamber after it has passed through a bar rack (that prevents large objects from entering the screening/dissolved-air flotation unit). The screen that is in the screen chamber consists of an open-ended screen drum which rotates in the screen chamber. Solids that accumulate on the drum are flushed from the screen. The solids can be removed from the treatment area either by flushing them to a nearby sanitary sewer or by dewatering and hauling them to a landfill site. A portion of the effluent from the screen chamber is diverted from the main flow by two pumps. The diverted flow from one of the pumps is used to wash the accumulated solids from the rotating screen while the diverted flow from the second pump goes into an air solution tank where it is saturated with air. The air-charged water is then routed to the inlet end of the flotation chamber where it mixes with the remaining water from the screen chamber. As the mixed water flows through the flotation chamber, the rising

air bubbles in the water attach themselves to particulate matter and float such matter to the waters surface. The floating matter is removed by mechanical skimmers and is routed to a nearby sanitary sewer. Chemical flocculating agents, such as ferric chloride, are added to the water in the flotation chamber in order to remove nutrients. Chlorine is also added to the water in order to provide for disinfection of the water before the treated water is released to the receiving stream.

The screening/dissolved-air flotation system was developed in 1967 when Rex Chainbelt, Inc. (now Envirex, Inc.) was awarded a contract by the U.S. Environmental Protection Agency to develop and to demonstrate a feasible method of treating combined sewer overflows. After consideration of various alternatives, a combination screening and air flotation process was selected as having the best potential. A demonstration facility was constructed at Hawley Road and Trenton Place in the City of Milwaukee near the outfall of an 8-foot, 6-inch by 5-foot combined sewer that served an area of about 500 acres. The demonstration facility was designed to treat up to 5 mgd of the combined sewer overflow. While the initial demonstration activities concentrated on performance with respect to the efficiency of the system in removing solids and in achieving chemical and biochemical oxygen demand reduction, subsequent research has shown that the addition of chemical flocculating agents can be utilized to effectively achieve nutrient removal.

The initial results of the Hawley Road demonstration facility, as operated through October 1969, were reported in summary form in a paper delivered at a symposium conducted by the U.S. Environmental Protection Agency, Water Quality Office,

in Chicago in June, 1970[1/]. The performance of the facility up to that time is summarized in Table 1. The data indicates that removals of about 50 percent of the BOD and between 50 and 70 percent of the suspended solids can be achieved without additional flocculants by the screening/dissolved-air flotation system.

Table D-1

CONTAMINANT REMOVALS BY
SCREENING AND AIR FLOTATION AT THE HAWLEY
ROAD DEMONSTRATION FACILITY,
MILWAUKEE, WISCONSIN:
(NO FLOCCULANTS UTILIZED)

Parameter	Removal by Screening*		Removal by Screening & Flotation*	
	Spring	Summer-Fall	Spring	Summer-Fall
Biochemical Oxygen Demand..........	23.4 ± 9.3	20.3 ± 6.5	48.4 ± 15.7	50.8 ± 12.5
Chemical Oxygen Demand..........	33.9 ± 10.7	22.4 ± 5.0	52.9 ± 8.7	53.4 ± 8.6
Suspended Solids..	28.8 ± 10.5	24.9 ± 9.8	53.7 ± 11.7	68.3 ± 8.4
Volatile Suspended Solids..........	28.2 ± 13.6	24.4 ± 13.2	51.0 ± 15.9	64.8 ± 10.0

* Removal as percent at 95 percent confidence level, screen openings 297 microns: surface loading 2.5 gpm/sq ft.

1/ Donald G. Mason, Rex Chainbelt, Inc., Milwaukee, Wisconsin. The Use of Screening/Dissolved-Air Flotation for Treating Combined Sewer Overflows, presented at the Symposium on Storm and Combined Sewer Overflows, June 22-23, 1970, Chicago, Illinois.

Table D-2

NITROGEN AND PHOSPHORUS REMOVALS
AT THE HAWLEY ROAD DEMONSTRATION FACILITY, MILWAUKEE, WISCONSIN
WITH UTILIZATION OF FERRIC CHLORIDE-1970

	Nitrogen			Phosphorus		
		Removal Efficiency			Removal Efficiency	
Run Number	Influent Nitrogen (Kjeldahl) (mg/l)	With Low Settling Rate[a/] (Percent)	With High Settling Rate[b/] (Percent)	Influent Phosphorus (Soluble) (mg/l)	With Low Settling Rate[a/] (Percent)	With High Settling Rate[b/] (Percent)
9	4.40	23	23	0.92	90	94
11	8.80	55	55	0.67	96	88
12	6.30	24	30	0.95	99	95
13	13.90	42	46	1.07	94	94
15	4.90	27	47	0.47	94	96
19	6.95	65	56	7.0[illegible]	96	86
20	3.40	41	44	0.73	92	75
21	2.40	25	25	0.77	85	89
22	19.50	76	72	2.46	60	46
23	8.70	30	39	2.73	89	94
24	4.90	27	22	0.74	42	41
25A	6.40	38	39	1.78	97	97

a/ 2.5 gpm/sq ft   b/ 3.75 gpm/sq ft   Source: Envirex, Inc.

The performance achieved at the Hawley Road facility confirms published results[1/] of a similar demonstration program conducted by the Rhodes Technology Corporation of Houston, Texas, in the City of Fort Smith, Arkansas. The data that is reported by the Rhodes Technology Corporation indicates that the range of reduction in pollutional parameters during dry- and wet-weather flow conditions, with and without the use of chemical treatment as an adjunct to mechanical separation, compares favorably to results normally achieved in secondary sewage treatment facilities with respect to removal of suspended solids and BOD and, with chemical additives, exceeds the performance of such facilities with respect to nutrient removal.

In July of 1973, Rexnord, Inc. was awarded a grant by the U.S. Environmental Protection Agency to study the screening/dissolved-air flotation system on a full scale for the treatment of combined sewer overflows. The site for this project was the City of Racine, Wisconsin. The system is operating at full scale and the results are being evaluated over a two year program. No data is available at the present time.

1/ Rhodes Technology Corporation, Dissolved-Air Treatment of Combined Sewer Overflows, Houston, Texas, January 1970.

The initial findings of the Hawley Road demonstration project were summarized in the paper as follows:

> Based on the data collected during the study and reported herein, it appears that the screening/dissolved-air flotation can be utilized as a successful alternate to sewer separation in some areas. Removals of BOD, COD, SS, and VSS in the range of 50-75 percent were recorded for the 30 overflows monitored to date. The solids removed from the overflows represented only about 1 percent (by volume) of the raw waste-water flow and had a concentration of 2 to 4 percent. The entire system is completely automated and requires a minimum of maintenance.

Operation of the Hawley Road demonstration facility after October, 1969 indicated that, with the addition of ferric chloride and polymers, substantial removal of soluble phosphorus and nitrogen was possible (see Table 2). The screening/dissolved-air flotation system achieved phosphate removals in excess of 85 percent for 19 of the 24 conditions studied. It should be noted that the performance variations shown in Table 2 may be attributed to the experimental nature of the facility. Improved, permanent installations should be able to consistently achieve higher levels of phosphorus and nitrogen removal than were achieved at the Hawley Road facility.

APPENDIX E

SEWER SEPARATION

Appendix E

SEWER SEPARATION

The cost of sewers separation has been investigated for several cities with combined sewers and has been reported in a 1968 study[1/] by the American Society of Civil Engineers (ASCE) under contract with the Federal Water Pollution Control Administration (FWPCA). The objective of the study was to evaluate the feasibility and costs of sewer separation using pressurized sewers, as conceived by Professor Gordon M. Fair of Harvard University. The cost of sewer separation with pressurized sewers was compared in the study with the costs of sewer separation utilizing conventional gravity flow systems and were found to be about 1.5 to 1.0 times as high. The costs of the conventional gravity flow system were determined for San Francisco, California; Milwaukee, Wisconsin; and Boston, Massachusetts.

Costs for sewer separation were also reported by the Environmental Protection Agency in a recent study[2/]. Approximate costs for 16 cities showed on average of $12,247 per acre served as of 1964.

1/ U.S. Department of the Interior, Federal Water Pollution Control Administration, Combined Sewer Separation Using Pressure Sewers, Feasibility and Development of a New Method for Separating Wastewater from Combined Sewage Systems, Water Pollution Control Research Series ORD-4, October 1969 by the American Society of Civil Engineers.

2/ EPA publication "Alternative Waste Management Techniques for Best Practicable Waste Treatment" March 1974.

The lowest cost of separation of sewers was in the Milwaukee test area. In a report[3/] jointly prepared by the ASCE staff and Greeley and Hansen, Consulting Engineers, Chicago, Illinois, the study area characteristics are reported to be as presented in Table E-1.

Table E-1

CHARACTERISTICS OF MILWAUKEE, WISCONSIN
STUDY AREA FOR HYPOTHETICAL APPLICATION OF
ASCE COMBINED SEWER SEPARATION PROJECT

Characteristic	Milwaukee, Wisc.
Extent of Gross Area	157 acres
Type of Development	
Present	Mainly Residential
Projected	Primarily Residential with large apartment complexes
Length of Combined Sewers	33,000 feet
Topography	Gently Sloping (El. 30 to 80 feet)
Population	11,300 (1966) 14,000 (1993)
Dwelling Units	3,500 (1966 Est.) 5,800 (1993)
Number of Service Connections	843 (1993)
Special Difficulties	Closely spaced buildings

The results of the study for the Milwaukee area are presented in Table E-2.

3/ Ibid.

The cost of conventional separation of sewers can be described as a function of the number of structures within the area and the areal extent of the service area. This cost can be expressed as follows:

$$\text{Cost} = \$12,080 \times A_1 + \$2,435 \times S$$

where A_1 = Area tributary to new sanitary collection system, area; and

S = Number of structures in service area

Table E-2

ADJUSTMENT OF CAPITAL COSTS FOR
PREPARATION OF COMBINED SEWERS IN THE
MILWAUKEE, WISCONSIN, STUDY AREA

Item	Cost in ASCE Study 3/	Adjusted Cost
In-house Separation	$ 912,000	$ 912,000
+ Escalation ENR Construction Index of 1200 to 2000	-	608,000
Area Collection	843,000	843,000
+ Escalation 1968 to 1974	-	562,000
Subtotal	1,755,000	2,925,000
Engineering & Contingencies		
at 25%	440,000	-
at 35%	-	1,024,000
Total	$2,195,000	$3,949,000
Cost per structure	2,610	4,700
Cost per Acre	14,000	25,200
Cost In-House (per structure)	-	2,435
Cost Area Collection (per acre)	-	12,080

3/ Ibid.

These costs are based upon construction of new separate sanitary sewer system and modification of existing house and building sewers to eliminate clear water to the sanitary sewers.

As seen from the above cost equation, the cost of separation will be substantially lower in areas where the number of structures is relatively low. Where S is low, the area tributary to the new sanitary collection system is probably less than the area tributary to the existing combined sewer system. Thus, the factors that tend to economically favor sewer separation are urban renewal, low density housing, redevelopment, large undeveloped areas, and proximity to the main interceptor.

Urban renewal programs that require demolition of existing structures reduce the cost of sewer separation. By razing structures connected to the combined sewer system, the connection of the new structures to the collection system would cost approximately the same whether the system were separated or left combined. Therefore, the cost term "$2435 x S" is reduced to zero. Redevelopment projects have the same effect as urban renewal projects. The proximity of an area to the main interceptor affects the cost of conveying separated storm water and sanitary sewage to the river interceptor.

Low density housing in combined sewered areas also favors sewer separation. Within an area, the lower the density of houses, then the smaller the cost of in-house separation. In addition to low density housing, there may be large areas without structures. Thus, the area that would be tributary to a new separate sanitary sewer system could be noticeably less than the area tributary to the combined sewer system. For a given size area, low density housing and large tracts of land without structures both favor sewer separation.